CONTROL OF GROUND WATER FOR SLOPE STABILITY BY TRENCHING METHOD

[illegible]

[illegible]
[illegible]
[illegible]

CONTROL OF GROUND WATER FOR SLOPE STABILITY BY TRENCHING METHOD

By

Mahwasane Tshiluvhu Lowani

Supervisor: Assoc Prof. Dr Nasiman Sapari

Dissertation submitted in partial fulfillment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil Engineering)

Jan 2009

Universiti Teknologi PETRONAS
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

CONTROL OF GROUND WATER FOR SLOPE STABILITY BY TRENCHING METHOD

By

Mahwasane Tshiluvhu Lowani

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfillment of the requirements for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved By:

(Assoc. Prof.Dr Nasiman Sapari)

UNIVERSITI TEKNOLOGI PETRONAS
TRONOH, PERAK
January 2009

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or person.

(Mahwasane Tshiluvhu Lowani)

ABSTRACT

Slope failure can be defined as the slope that collapses due to weakened self-retaining ability of the earth under the certain influence such as rainfall, earthquake and behavior of soil itself. Slope movement depends on two main factors which are slope steepness and slope stability. Many slope failures have taken up during or after a period of rain and its can be concluded that rainfall is the main cause of slope failure. This report presents the results from site investigations and soil tests carried out at the campus of Universiti Teknologi PETRONAS (UTP), where a landsliding had occurred behind building 13. Experiment for soil characteristics was conducted in the laboratory and a model on the influence of rising groundwater level towards the slope stability was designed and tested. The model used was a rectangular tank with dimensions of 1m length of 0.5 m and width of 0.4 m deep. The slope angle was about 33^{o} representing angle of a cut slope. PVC pipe was used as a trench and also for standpipe reading. Different water level was used to bring about different groundwater level that might occur on site. The influence of different water level of groundwater was examined by determining the moisture content of the soil as well as the level of water in the standpipe. It was found that the presence of the Trench reduces the level of moisture on the soil even when the height of the groundwater table is at the same level as the soil. The moisture content was always below 24% above the trench while the soil below the trench reached the liquid limit of about 42%. This results differs from those recorded earlier from previous studies done without the trench provided, from their results it was obtained that as water level become higher than the toe the moisture of the soil increased to the liquid limit level this may cause slope failure.

ACKNOWLEDGEMENTS

In this opportunity, the author would like to express her appreciations to those who had assisted in completing the report for Final Year Project 2 (VAB 4034). Highest appreciation express to the project Supervisor, Assoc. Prof. Dr. Nasiman Sapari for leading me towards the right direction and guidance throughout the project completion. The author's greatest gratitude also goes to Mohd Zaini b hashim from Universiti Teknologi PETRONAS who is a Technician for his cooperation in sharing data and helping in the project. The efforts of those involved in preparing this report are gratefully acknowledged.

TABLE OF CONTENT

LIST OF ABBREVIATION

PVC Polyvinyl Chloride

FOS Factor OF Safety

XRD X-Ray Diffraction

SEM Scanning Electron Microscope

BH Bore Hole

SPT Standard Penetration Test

Y_{max} Maximum design depth

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

This chapter introduces the reader to the research project it objective and the slope of the studies that was conducted in controlling ground water movement for slope stability.

1.1 Background Of Study

Slope Failure is a common geotechnical problem, experienced world wide. Slope failures are caused, in general, by natural forces, human misjudgment and activities and burrowing animals. Erosion, Rainfall, Earthquakes, Geological features, External loading, Construction activities and Rapid drawdown are the main factors that provoke slope failure (Bubhu, 2007).

Slope stability depends on geometry, soil characteristics and the forces to which it is subjected to both internally and externally. A key normally neglected is the seasonal change of the groundwater due to the change of the pore water pressure. The fluctuation of the pore water pressure depends more on the intensity of rainfall that a certain area experiences. High rainfall causes an increase in pore water pressure that in most cases result in slope failure. The change in external water level without allowing the time needed for the drainage of the slope soil is called sudden or rapid drawback (RDD) (Berilgen, *2007).*Variability of site conditions, local geology and rainfall contributes significantly to the uncertainties of landslide hazards.

Land sliding is one of the slope movements that can cause extensive damage to structures and may be more expensive to stabilize when the impact is on developed property. D'Acunto and Urciuoli (2006) consider for slope instability in clayey soils the trenches are the most widely used to control the problems. The action of the drains reduces pore pressure in the subsoil and consequently increases effective stresses and soil shear strength (D'Acunto and Urciuoli, 2006). For instance, after a heavy rainfall water will percolate down the slope but if there are trenches, the soil will not be saturated for long since the pore water pressure will decrease due to the flow of water into the trenches. Thus, the strength will increase due to the contact of the soil particles. The higher the moisture content, the lower is the friction between the soil particles. Trenching is less costly than other types of control works and it is suitable for a large number of situations, even when the landslide is very deep and structural measures are inadequate (D'Acunto and Urciuoli, 2006).

1.2 Problem Statement

In the Universiti Teknology PETRONAS (UTP) campus area, land sliding has occurred behind building 13 as shown in figure 1.1 and appendix 6. The hill side of Sri Iskandar areas consists of, residual soil from the weathering products of sedimentary rocks. The soil where the campus is located, mainly originated from the weathering of silt and sand stone. The sliding slope was provided with surface drainage to reduce erosion, it is believed that there are two possible causes of the landslide namely:

1. Excessive surface infiltration of water from a surface broken drain
2. Increase in groundwater level that lead to high pore water pressure in the soil

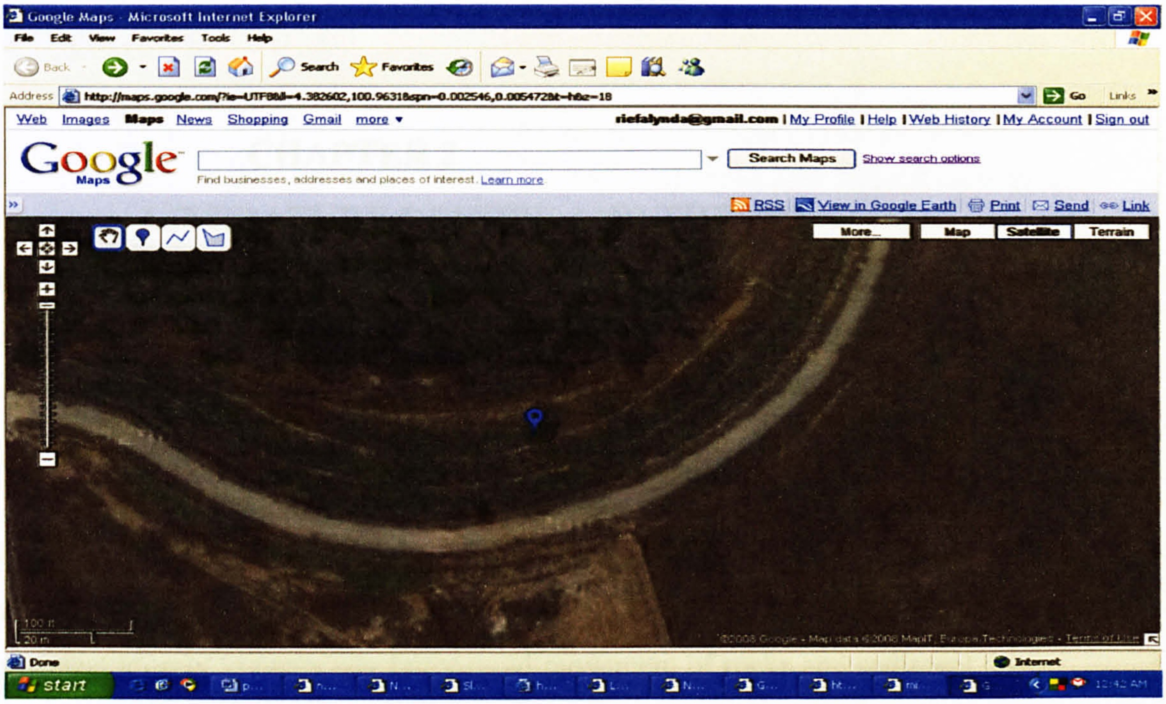


Figure 1.1: Plan view of site of slope failure

1.3 Objectives

The objectives are as below:

- To identify the properties and characteristic of the residual soil of Sri Iskander Area
- To examine the effectiveness of Trenching and under ground drainage in decreasing moisture level of the soil
- To determine the new Factor of Safety (FOS) using the slope W software after reducing the moisture.

1.4 Scope Of Study

The scope of study comprises of field investigation, soil sampling and development of a laboratory model. The model was tested to examine effectiveness of trenching method in reducing soil moisture.

CHAPTER 2

LITERATURE REVIEW AND THEORY

This chapter summarizes the literature review and theory involved for carrying out the project. Important aspects of the project are briefly explained, these include, theory on landslide, cases of landslide, causes and methods for ground water control.

2.1 Theory of Landslide

Landslides are described as a mass movement of soil or rock that involves shear displacement along one or several rupture surfaces. Although the term landslide is often used somewhat loosely to mean any fairly rapid movement of rocks and sediment down slope.The shear displacement along a distinct rupture distinguishes landslide from other types of soil or rock movement such as falls, topples or flow. Land slide can be triggered by an increase in shear stress(i.e. Removal of lateral support, apply of surcharge at the end of landslide, apply a lateral pressure and apply of Vibration force) or a reduction in shear strength (i.e. Natural weathering of soil or rock, development of discontinuities and increase of moisture content). For design slope to be stable, the factor of safety cannot be too low. Decrease in driving force, increase in resisting forces, or rebuild the slope are three basic approaches that can be used to increase the factor of safety **(Day,1999). According to D'Acunto** and Urciuoli (2006), a Factor of Safety (FOS) =1.2-1.3 is considered as a large enough factor for safety.

Figure 2.1 shows a cross sectional area of scarp and deposits from landslides. Rock breaks away from the wall along a curved fault, and first slides downhill as fairly solid slump blocks. A portion of the slip face is left exposed as the landslide scarp or headwall, the outflow liquid is the debris apron.

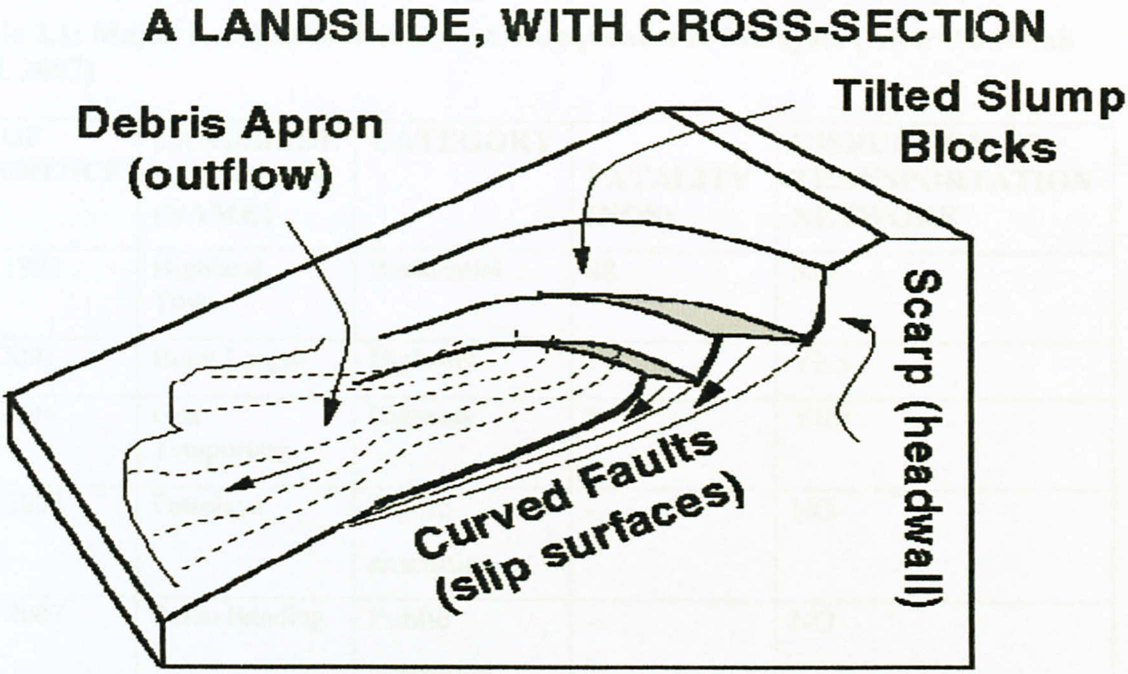


Figure 2.1: Landslide with cross section

2.2 Historical cases of Landslide in Malaysia

Major landslides occurring within the area of infrastructure seldom result in loss of lives compared to those occurring in residential areas. However, major landslides that occurred within infrastructures have resulted in great economic loss to the public and business due to disruption to the transportation network and property damage (Abdullah et al. 2007).

The following table summarizes some of the major landslides with their consequences: -

Table 2.1: Major Landslides with their Consequences in Malaysia (After Abdullah et al. 2007)

DATE OF OCCURRENCE	LANDSLIDE LOCATION (NAME)	CATEGORY	FATALITY (NOS)	DISRUPTION TO TRANSPORTATION NETWORK
11 Dec 1993	Highland Tower	Residential	48	NO
26 Oct 2003	Bukit Lanjan	Highway	-	YES
12 Oct 2004	Gua Tempurung	Highway	1	YES
23 Mar 2007	Putrajaya	Public amenities	-	NO
13 Nov 2007	Pulau Banding	Public amenities	-	NO

The details of the two major landslides namely the Bukit Lanjan and Putrajaya landslides are discussed below

2.2.1 Rock Slope Failure at Bukit Lanjan, 2003

On 26 Nov 2003, a massive rock slope failure occurred at Bukit Lanjan Interchange which is part of the New Klang Valley Expressway (Asbi et al. 2007). The failure occurred immediately after a period of heavy rainfall. The substantial large volume of rock debris (approx. 35,000m^3) that came to rest on the expressway blocked the expressway completely and forced the entire stretch of the expressway to be closed for 6 months for rehabilitation works. Immediately after the failure, the Highway Concessionaire commissioned site investigations that included surveys, geological mapping, deep boreholes and laboratory tests to assess the likely causes of failure and also to provide geotechnical information required to design for rehabilitation of the failed slope. From the site investigation results, it was inferred that the rock slope failure was a complex wedge type failure due to jointing system of the rock in the area.

2.2.2 Slope Failure at Putrajaya, 2007

On 22nd March 2007, a massive slope failure occurred at Precinct 9, Putrajaya which twenty-three vehicles were buried in this landslide and forced about 1,000 residents to vacate their homes at 4.30am. This slope failure involved a 50-metre high hill with a man-made slope about 45 degrees which was located about 10 meters from the 15-storey apartment. It had been raining heavily in Putrajaya since the evening of 21 March 2007 till the early morning of 22 March 2007 before the slope failure happened. Figure 2.2 shows the collapsed slope with buried vehicles.

Figure 2.2: Slope Failure at Precinct 9, Putrajaya, 2007

2.3 Causes of Landslide

Landslides occur when the stability of a slope changes from a stable to an unstable condition. A change in the stability of a slope can be caused by a number of factors, acting together or alone.

According to Chigira,et al. (2003) the causes are:

Natural causes:

- groundwater pressure acting to destabilize the slope
- Loss or absence of vertical vegetative structure, soil nutrients, and soil structure.
- erosion of the toe of a slope by rivers or ocean waves
- weakening of a slope through saturation by snowmelt, glaciers melting, or heavy rains
- earthquakes adding loads to barely-stable slopes
- earthquake-caused liquefaction destabilizing slopes
- volcanic eruptions

Human causes:

- vibrations from machinery or traffic
- blasting
- earthwork which alters the shape of a slope, or which imposes new loads on an existing slope
- in shallow soils, the removal of deep-rooted vegetation that binds colluviums to bedrock

A study of the causes of landslides such as design errors, construction errors, design and construction errors, geological features and maintenance had been carried out by Gue and Tan (2006) based on 49 investigation cases of primarily large landslides on residual soils. The results of the study are shown in Table 2.2

Table 2.2: Causes of Landslides (Gue & Tan, 2006)

LANDSLIDES	NUMBER OF CASES	PERCENTAGE (%)
Design Errors	29	60
Construction Errors	4	8
Design & Construction Errors	10	20
Geological Features	3	6
Maintenance	3	6
Total	49	100

2.4 Methods of rectification and soil improvement

Stabilizing the soil on the slope is the answer to landslide. The most important modification to be made is the hydraulic modification because the slope is not stable due to flow of surface water or groundwater. The ways to control a slope includes:

2.4.1. Baffles or barriers

Baffles or barriers are obstruction devices that slow down or divert water from flowing directly downhill. They consist of partially buried stone or timbers (laid parallel to the slope). These barriers work best for lesser slopes.

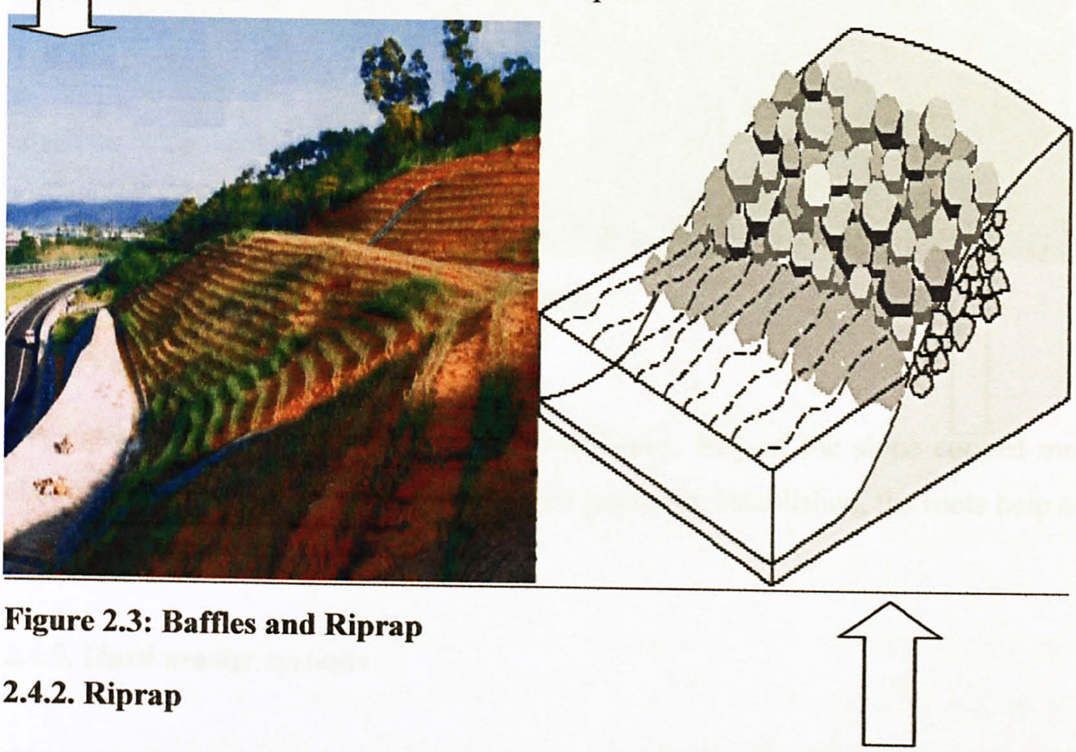


Figure 2.3: Baffles and Riprap

2.4.2. Riprap

Riprap is rough, loose stone (at least 6"-8" diameter) usually granite. Stone is imbedded into or spread loosely onto the slope. Riprap also slows and diverts flowing water. The areas between the stones can be planted with a variety of ground covers or rock garden plants.

2.4.3. Terraces

Terraces are stair-step up the slope. The flat surfaces allow you to plant on the terraced levels. Terraces allow water to soak in instead of running off. Timber, stone, concrete or precast concrete block are used to build the retaining walls.

Figure 2.4: Terraces and vegetation plants

2.4.4. Vegetation plants

Vegetation plants are often **used to control slopes. Any of the slope control methods** above can be planted with vegetation. When plants are established, the roots help anchor the soil.

2.4.5. Hard amour systems

This hard amour systems method is suitable for soil banks or slopes exposed to constant concentrated flows, currents, or waves that cannot support vegetation.

2.5.6. Perforated piping

Perforated piping is a means of drainage through a tube which is inserted into the slope at a particular angle to drain out water from within the slope and prevent water retention in the slope.

2.5 Ground Water

Hill slope morphology, material properties, and hydraulic heterogeneities influence the role of groundwater flow in provoking slope instability. The influences of these factors were examined by employing the elastic effective stress model and Coulomb failure potential concept by Turner and Schuster (1996). Gravity-driven groundwater flow strongly influences the shape of equilibrium hill slopes, which was define as those with uniform near-surface failure potentials. For homogeneous slopes with no groundwater flow, equilibrium hill slope profiles are straight; but with gravity-driven flow, equilibrium profiles are concave or convex-concave, and the largest failure potentials exist near the bases of convex slopes. In heterogeneous slopes, relatively slight hydraulic conductivity contrasts of less than 1 order of magnitude markedly affect the seepage force field and slope failure potential (Saxov and Nieuwenhuis, 1982).

The effect of slope instability can be mitigated by using both artificial and natural methods. Among the natural methods covering the slope surface with vegetation is one of the most effective techniques. To analyze the contribution of vegetations in erosion control and slope stability, one needs to think of its hydrological, biological and mechanical role (Brunsden and Prior, 1984). The mechanical contributions arise from the physical interactions of either the foliage or the root system of the plant with the slope. The hydrological mechanisms are those processes of water use and movement in the slope when living plants exist in the soil. In addition, the existence of plants on the ground affects the biological process in the soil underneath and also in the surrounding, which may have both positive and negative roles in slope stability and erosion control (Zaruba, Mencl, 1969).

The stability of any slope depends on the strength of the soil material comprising of slope and slope geometry. Less can be done to change the slope geometry for the stability of natural slopes. However, more can be achieved if appropriate soil bioengineering techniques are used (Brunsden and Prior, 1984). When properly installed and maintained, vegetation can protect slopes by reducing erosion, strengthening soil, and inhibiting landslides which increase general slope stability.

According to Ludwig, et al. (1993) , groundwater can be a long-term 'reservoir' of the natural water cycle (with residence times from days to millennia), as opposed to short-term water reservoirs like the atmosphere and fresh surface water (which have residence times from minutes to years).The effect of groundwater in hydrological cycle is shown in figure 2.5 below.

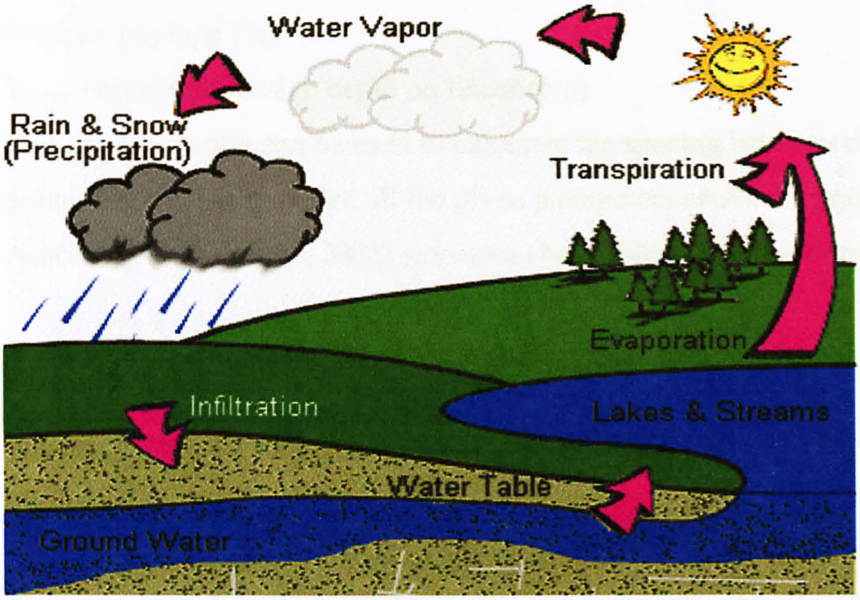


Figure 2.5: groundwater plays role in Hydrological cycle

2.5.1 Determining maximum design depth

Darcy 's law in conjunction with the law of continuity can be developed to an equation for calculating spacing between the pipes in leachate-collection system (Vesilind, &Worrell, 2000).

$P=2Y_{max}/[(q/K)\ [(K\tan^2\partial/q)+1-(K\tan\partial/q)\ ((\tan^2\partial+q/K)^{0.5})]]$

Where:

q=design storm (cm/s)

K=hydraulic conductivity (cm/s)

∂=drainage slope (%)

Y_{max} =maximum design depth on linear (cm)

This same equation can be used to calculate the spacing between drainage pipes to lower groundwater level provided all the given parameters above are known.

According to Waltham (2002) slopes can be stabilized by one or more of the followings:

I. Modifying the slope profile

Increasing stability can be done by adding material below the neutral line if possible;

II. Supporting or anchoring the existing profile

Support can be done by retaining walls, concrete walls, dental masonry, gabion wall and sprayed concrete(shotcrete) or by anchoring such as Rock bolts, grouted dowels, bored piles ,geotextile, grouting , vegetation and rock anchors;

III. Improving or draining the slope material

This is the most economical method to stabilize large slides in natural slopes. Pore water pressure is critical to slide stability, so drainage is usually very effective.

2.6 Factor of safety

Slope stability is a critical and fundamental element of Geotechnical Engineering involving the assessment of existing natural slope and the intelligent design of man-made slopes. It is based on the interplay between two types of forces which is driving force and resisting force.

Gravity is the main driving force in most land movement while material's shear strength for resisting force and it acts on all objects on Earth's surface. Slope angle, climate, slope material, and water contribute to the effect of gravity; while for resistance force or the shear strength is the function of cohesion (ability to attract and hold each other together) and internal friction (friction between grains within material). The relation between resisting force (gp) and driving force (gs) can be elaborate more in term of Factor of Safety (FOS) and it is shown in figure 2.6. The factor of safety (FOS) is a ratio between resisting force to driving force:

$$SF = \frac{\text{Resisting Force (gp)}}{\text{Driving Force (gs)}}$$

As per Bishop's Simplified Method, (Bishop, 1955)

$$FS = \frac{\sum\left[(c\beta + W\tan\phi)\left\{\cos\alpha + \frac{\sin\alpha\tan\phi}{FS}\right\}\right]}{\sum W\sin\alpha}$$

If SF > 1, then SAFE

If SF < 1, then UNSAFE

Figure 2.6: Components of gravity oriented parallel (gs) and perpendicular (gp) to the slope for gentle and steep slopes

CHAPTER 3

METHOLODOGY

The method used in this study involves both field and laboratory works

3.1 Field investigation and soil sampling

Field investigation and soil sampling was conducted by a consultant and the results were recorded. On December 2008, a consultant was hired to investigate the slope failure on the site of this study area. The soil investigation works were carried out to obtain geotechnical engineering data of the subsoil for the design corrective of the slope and the engineering control during the construction of the proposed project. Several tests were conducted including:

Mackintosh probe test: The main objective of this test is to determine the possible weak soil strata. From the soil investigation report made from the study there were some weak layers found in the soil that occurs when there is a highly significant change in the depth of the penetration. The Mackintosh Probe is a lightweight and portable penetrometer. It is a considerably faster and cheaper tool than boring equipment especially when the depth of exploration is moderate and the soils under investigation are soft or loose (Fakher et al. 2006).

Borehole Tests: The objective is to get the information regarding soil conditions below the surface. The report indicates the soil type existing in this area has more or less the same characteristics as those determined in FYP1 from the laboratory. Soil found in the top possessed loose type of soil and that lying beneath possessed a very dense characteristic

Standpipes: The use of the stand pipe is to measure the level of water. Two stand pipes were installed one at the top (3rd tier) and the other one at the bottom (1st tier). Tiers are shown in appendix 7.

Additional investigations were conducted together with soil sampling for further analysis on the soil characteristics.

3.2 Soil Lab Analysis

Slope around UTP campus was investigated, soil sample was analyzed and laboratory work conducted. The table below indicates the test, objective and apparatus for some of the experiments. All the procedures conducted for the experiments were according to BS 1377, BS 1337.

Table 3.1: The method of soil analysis

Test	To determine	Apparatus
Oven-Drying Method	Moisture content in soil	Drying oven, moisture content tins, Electronic balance
Vane Shear Test	The Shear strength of soft cohesive soil	Laboratory vane apparatus
Specific Gravity	The value of particle Density	Pyknometer, thermometer, electro balance, glass rod
Sedimentation by hydrometer	Particle distribution according to BS 1377	Hydrometer graduated, stop watch, constant temp bath,etc.
Plastic Limit and Liquid Limit	The plastic limit and Liquid limit of soil using penetrometer	Flat glass, two spatulas, rod compactor ,Two spatulas, cone penetrometer, glass plate,etc.
Permeability Test	The coefficient of permeability	Permeameter cell, vertical adjustable reservoir tank, stop watch,etc.

3.3 Development of Laboratory Model and analysis of soil provided with trenches

Soil was collected from the lowest part of the sliding slope for safety reasons, since the soil was wet due to the frequent occurrence of rainfall in this area. It is believed that the soil sample collected is from the 2nd tier of the sliding slope but rolled down the slope. The soil was analyzed and the results were recorded. In order to determine the factor of safety of the slope several tests were conducted in the laboratory. A laboratory model was developed for the study on the relationship between rising water table on slopes that are provided with trenches. Below is the diagram showing the flow of works that were carried out involving laboratory model.

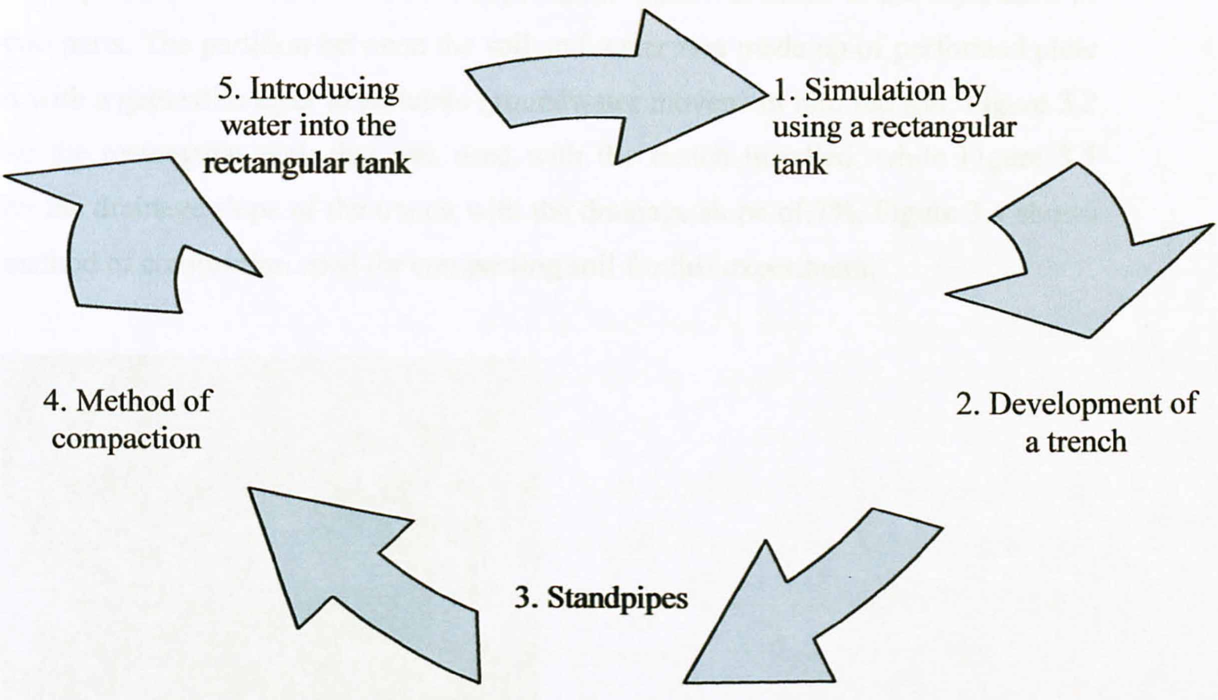


Figure 3.1: Steps followed for the development of the Model

3.3.1 Simulation by using a rectangular tank

A rectangular tank with the following dimensions 1m*0.4m*0.6m as shown in figure 3.2 below, was used to construct a physical slope model. Mass of the soil to be compacted into the tank was computed using the bulk density obtained from the laboratory analysis with a slope angle of 33°. The angle of the slope was determined by measuring the angle of the real slope (behind building 13) by using a compass (it was found to be 33°). This is in accordance with JKR specification for residual soils of 1:1.5. The average of the two was considered as the angle of the slope. The tank was divided into two part one with soil and the other one with water as shown in the figure 3.2 below. The larger portion which is about 0.7 m is where the soil sample was placed and the other portion is for water. For this experiment 0 cm was taken at the separation of the two parts. The partition between the soil and water was made up of perforated plate lined with a geotextile layer to simulate groundwater movement into the soil. Figure 3.2 shows the rectangular tank that was used with the trench installed, while Figure 3.5 shows the drainage slope of the trench with the drainage slope of 7%. Figure 3.4 shows the method of compaction used for compacting soil for this experiment.

Figure 3.2: Rectangular tank **Figure 3.4: compaction in progress**

Figure 3.5: 0.07% slope of the (∂) drainage slope

3.3.2 Development of a Trench

For simulating a Trench, a polyvinyl chloride (PVC) pipe was used with a diameter of 23 mm and placed at an angle to the tank. Two PVC pipes were used of the same diameter and length at each side of the tank. This was done to allow the movement of water from one point to another since the outlet of water had a slightly higher gradient than the inlet (i.e. From the other side of the tank where there is water). As we all know it is almost impossible for water to travel from an area of low point to high therefore gradient was provided on the PVC pipe.

A polyvinyl chloride (PVC) pipe is made from a plastic and vinyl combination material. The advantage of this pipe: pipes are durable, hard to damage, and long lasting. A PVC pipe does not rust, rot, or wear over time. For that reason, PVC piping is most commonly used in water systems, underground wiring, and sewer lines.

A distance was made from the separation of the two parts of the taken i.e. the zero point and the location of the PVC acting like a drain this was done so as to ensure that there is a specific time allowance that water will be given to flow within the soil before reaching the simulated drain where water will flow into. The distance from zero was arbitrary chosen as 10 cm away, from the sides of the tank a certain distance was left for it to be filled with soil which was about 2.5 cm on each side of the pipe.

In making this PVC pipes porous drilling had to be done, by using a drilling machine with a driller having diameter of 7.5mm. In each pipe about 60 holes were drilled to allow easy flow of water through it.

The PVC pipes are designed to allow the movement of water only, to prevent soil from seeping in the pores something had to be inverted that can allow the movement of water but prevent movement of other things can be soil or any other substances that might be there. According to Lance Brown Geotextile are permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain. Typically made from polypropylene or polyester, geotextile fabrics come in three basic forms: woven (These cloth-like fabrics are formed by the uniform and regular interweaving of threads with a regular visible construction pattern) and Non woven (These felt-like fabrics are formed by a random placement of threads in a mat and bonded by heat-bonding, resin-bonding or needle punching without any visible thread pattern.

In this experiment the Non-Woven geotextile was used. There were other permeable fabrics that could be used like mosquito net but due to the soil type this option was not totally applicable.

3.3 3. Standpipes

PVC pipes were used as stand pipes, piezometers for the monitoring of groundwater levels through a borehole. This plays a very significant role for the determination of the level of the groundwater as the water in the tank is raised to a specific level. Three standpipes were developed 1st, 2nd and 3rd stand pipes each with different height at distances 10cm, 33 cm and 53 cm respectively away from 0 cm which was specified above. The stand pipes were installed in every layer of soil.

3.3 4. Method of compaction

ASTM D698 - Standard Test Method for Laboratory Compaction Characteristics was followed in compacting the soil. This simply says apply 27 blows per layer for 3 layers of soil (Bowles, 1995). Since the soil consists of sand, silt and clay self compaction was conducted using a hammer with diameter if 5.5 cm raised 0.6 m from the sample at each blow. This is shown in figure 3.4 above. Three layers were made namely layer 3,layer 2 and layer 1 from the top to the bottom.

Air dried undisturbed sample from the site was used for this particular experiment. This was done to simulate or bring almost the same environment that is occurring in the site.

3.3.5. Introducing water into the tank

Water was introduced in the tank behind the slope and was made constant at each level for two days. Starting from the lowest layer, water level was incrementally increased from 7cm, 14cm then lastly 21 cm. It was kept for 2days at each level of increment, it was observed that the drain was functioning accordingly since it was able to remove water from the soil. This different water level increment is shown in figure 3.7 respectively as drawn using AutoCAD. Keeping the water flowing for two days needed a constant amount of water that was introduced into the tank, and again keeping the height constant monitoring had to be made since the level cannot be automatically be constant an hour after the water level is constant water that is being drained out had to be collected for a period of an hour and results recorded. The stand pipe readings were also recorded at the same time. The figure above shows a simple illustration of the whole model.

After two days of allowing the experiment to run, a quantity of soil was collected from each layer in order to obtain the moisture content. To avoid unbalance of mass the amount of soil removed was replaced again. The Reading on the stand pipes was taken again to ensure accuracy on results. The tap was closed (i.e. no more water introduced into the tank), the water level dropped until there was no more water, water was allowed to flow for days until there was no water being discharged and until the stand pipes read zero meaning there is no more water flowing. After this the Ground water table was raised to another height, this was continuously done until all the height of groundwater was done.

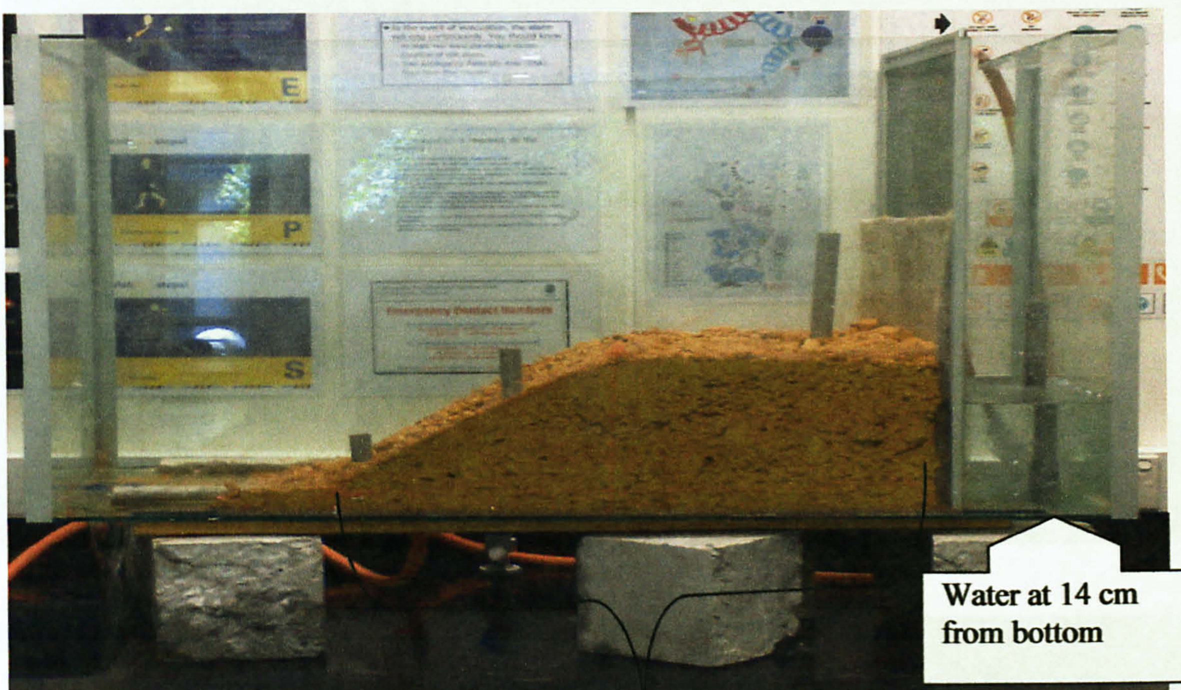


Figure 3.6: Model of the experiment

Figure 3.6 shows the actual model when water level was increased to 14 cm and monitored at that level for 2days.

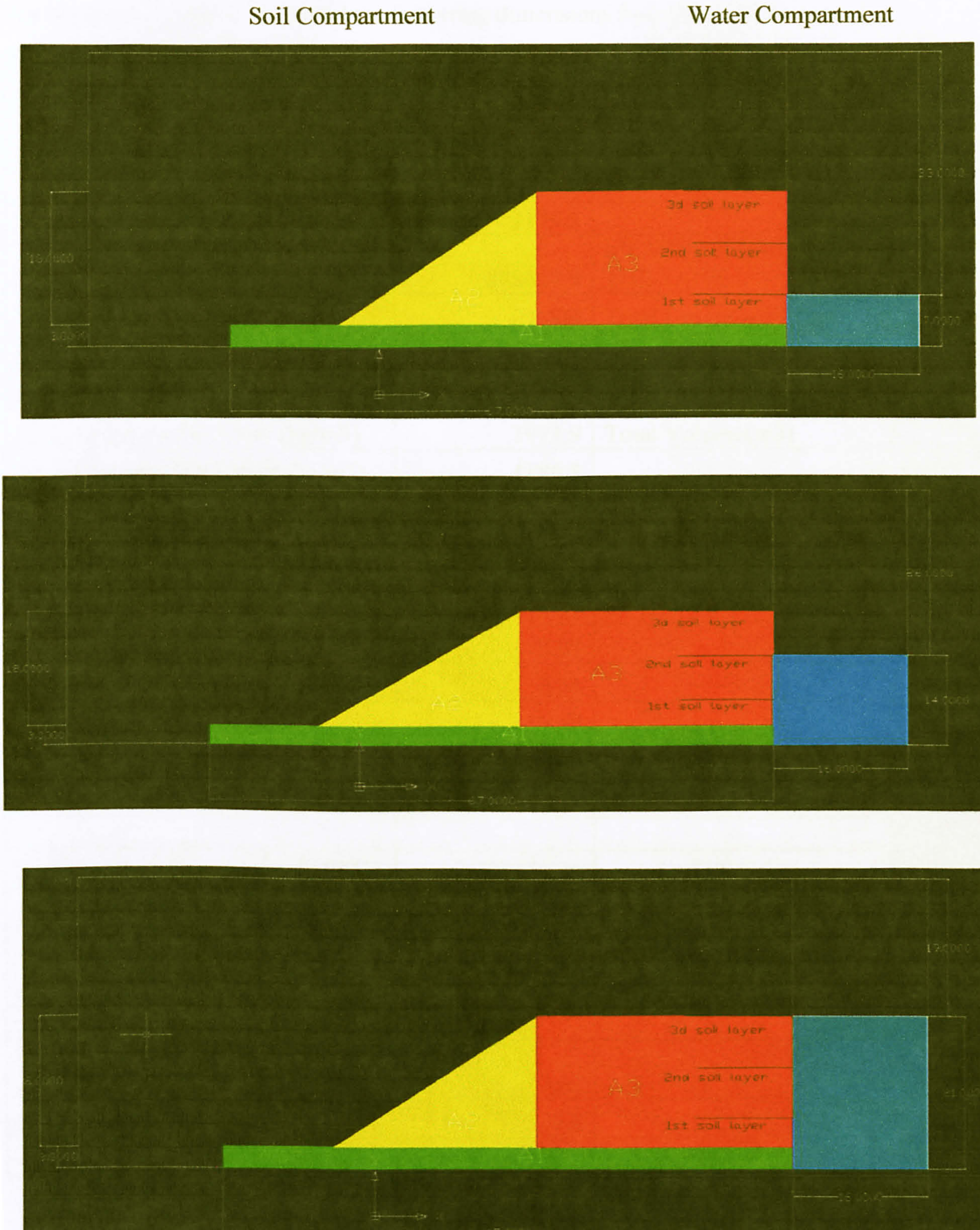


Figure 3.7: Different water levels 7 cm, 14cm and 21 cm from the model

3.4 Measurements used for the model development

For the development of the model the following dimensions from the table were used.

Table 3.2: Measurements used for the model development

mass of cylinder alone (g)	952.1		
diameter of cylinder (mm)	94.4		
Height of cylinder (mm)	127.0		
Mass of wet soil + cylinder (g)	2710.4		
Dry mass of soil and cylinder (g)	2179.3		
Volume (mm^3)	888627.9	MEASUREMENTS OF THE TANK	
Volume (m3)	0.0009	Total Length(m)(L_T)	1
Mass of wet soil only (g)	1758.4	Total Breath(m)(L_T)	0.4
Mass of dry soil only (g)	1227.3	Total Height(m)(H_T)	0.6
(D_w) Density of wet Soil (kg/m3)	1977.9	Total Volume(m3)	0.24
(D_d) Density of Dry Soil (kg/m3)	1380.5		
AREA CALCULATION			
Total Height of soil sample (m)	0.21		
L_A (m)	0.67		
B_A (m)	0.03		
B_B (m)	0.24		
H_B (m)	0.18		
L_C (m)	0.3		
B_C (m)	0.18		
$AA= L_A*B_A$	0.02		
$AB= 0.5B_B*H_B$	0.02		
$AC= L_C*B_C$	0.05		
(A_t)Total Area= AA+AB=AC(m^2)	0.10		
(V_s) Volume occupied by soil= A_t+H_T	0.02		
Mass of soil needed (Kg)= V_s*D_w	39.8		
44kg of soil was used			

3.5 Analysis of Slope stability using SLOPE/W software

Slope stability analysis is one of the process on determine the stability of slope for a given load. On this analysis using SLOPE/W software, data requirement that need to be gathered is the soil profile which is:

- Soil types
- Soil Properties
- Properties of slope (ex: slope height)
- Shear Strength
- GW Table
- Unit weight

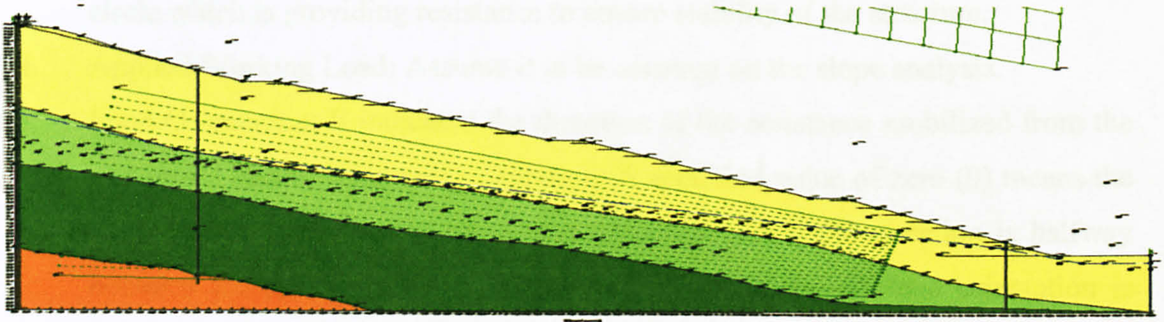


Figure 3.8: Analysis using SLOPE/W

The main thing that needs to be analyzing using SLOPE/W is to determine the slope stability is factor of safety (FOS) where on this software, many methods can be used to determine FOS:

- Ordinary or Fellenius
- Bishop's Simplified
- Janbu's Simplified
- Spencer
- Morgenstern-Price
- Corps of Engineers
- Lowe-Karafiath
- GLE (General Limit Equilibrium)
- Finite-Element Stress

There are 2 cases study that have been analyzed which are:

a. Case 1: Natural slope/ Slope failure state

b. Case 2: Slope with groundwater controlled i.e. with horizontal drain.

From the 2 cases, the main things that have been discussed were global failure and local failure on the analysis.

As for cases 2, Geo-fabric reinforcement load had been installed for the remedial part and data that need to be keyed in are as follows:

I. Working load: As fabric being used for geo-grid, no value need to be key-in as the parameter.

II. Bond Length: This length is automatically calculated by the software (not required as input). It actually means what is the geo-grid length behind the slip circle which is providing resistance to ensure stability of the structure.

III. Applied Working Load: Assume it to be constant on the slope analysis.

IV. Load Orientation: It indicated the direction of the resistance mobilized from the geo-grid. Usually it in axial orientation. A specified value of zero (0) means the direction is parallel to the slice base while 0.5 means the direction is halfway between the axial direction and slice base inclination. As load orientation is equal to 1, it means the reinforcement is parallel to the slice base.

V. Bond Resistant: The value of bond resistant are derived from soil properties. Bond resistant (F/L2) = (c + overburden stress x tan Ø) x interface factor.

VI. Shear Load: Usually ignored in the design (assume 0) as it is not required for fabric.

VII. Apply Shear Load: As shear load (VII) been assumed to zero, no shear load since shear load is not required for fabric.

Table 3.3: Detailed information for case 2

Key In Parameter	Value	Key In Parameter	Value
Working Load	0	Applied working load	constant
Load Orientation	0	Bond resistant	5.624
Reinforcement Load Max	150kPa	Shear Load	0
Applied shear load as	No shear load		

3.6 Health Safety & Environment

3.6.1 Activity

- Collect soil sample
- Testing soil characteristics

3.6.2 Hazard

3.6.2.1 Geotechnical Laboratory

- *Staggered objects on the floor*

- Equipments that are used by students are normally placed carelessly on the floor along the walking path of other students.

Example: soil sample, Drying oven, Mechanical sieve shaker.

- *Machinery*

- Some apparatus has to be manually held to avoid any errors that can occur when the experiment is being conducted.

Example: riffle box, test sizes

3.6.3 Prevention

- *Staggered objects on the floor*

If possible, students should wear safety boots or shoes that completely cover the foot.

- *Sharp objects*

always use protective gloves.

- *Machinery*

When handling heavy machinery, make sure not to work alone.

- *Electricity*

Do not operate electrical equipment and plug points with dirty or wet hands.

Chapter 4

RESULTS AND DISCUSSION

This section is enclosed with results obtained from soil characteristics, results from the previous studies by Sapari, et al. (2008) where Trenching was not provided, lab model studies and results recorded from the soil investigation report which was done by a consultant.

4.1 Soil characteristics analysis from lab

4.1.1 Determination of moisture content

a) Laboratory Results

Moisture content of the soil specimen collected from the landslide area is shown, as a percentage of dry soil mass

Table 4.1: Moisture content of the soil sample

Container NO:		1	2	3
Mass of wet soil+conteiner(m2)	g	88.1	79.8	70.2
Mass of dry soil+conteiner(m3)	g	72.94	65.47	65.56
Mass of container(m1)	g	21	20.99	20.33
Mass of moisture(m2-m3)	g	15.16	14.33	4.64
Mass of dry soil(m3-m1)	g	51.94	44.48	45.23
Moisture content(w)= (m2-m3)/ (m3-m1)*100%		29	32.2	10.6

b. Discussion

From the results in table 3 above it can be seen that the average moisture content is 24% which is high, this is an indication that soil from the site is wet and contains high amount of water.

4.1.2 Vane shear test

a) Laboratory results

deflection of spring=55^0=Θ_f

rotation of vane=11^0

rotation of spring mounting=66^0

M=66*0.199=13.134N.mm/degree

Where M=torque applied to shear

from this the vane shear strength of soil, Tv

Tv=(M/4.29)Kn/m2=(13.134/4.29)=3.06 Kpa

b) Discussion

The Vane shear strength was calculated, it did not take a long time for the soil sample to fail. This implies that the soil is not strong. From appendix 8, this type of soil was found to be very soft with shear strength of 3.06 Kpa which is $\leq$ 12.5 kpa the visual identification of this soil is that it can exude between fingers. According to Kehew (2006) blunt end of a pencil-size item makes deep penetration easily.

4.1.3 Particle Density

a) Results

Calculations of the particle density are provided in the table below, with all the masses recorded in grams.

Table 4.2: particle density

Particle density test		
initial mass of soil		400 g
mass of Jar+jar glass+plate+soil+water(m3)	g	1826.5
mass of Jar+jar glass+plate+soil(m2)	g	955
mass of jar+gas jar+plate+water(m4)	g	1563.6
mass of jar+gas jar+plate (m1)	g	538.5
mass of soil (m2-m1)	g	416.5
mass of water in full jar (m4-m1)	g	1025.1
mass of water used (m3-m2)	g	871.5
Volume of soil particles=(m4-m1)-(m3-m2)	ML	153.6
Particle density Ps=(m2-m1)/(m4-m1)-(m3-m1)	Mg/m^3	2.7
Average value ps	Mg/m^3	2.7

b) Discussion

The specific gravity was calculated and it was found to be 2.7 Mg/m^3 which is an acceptable value since it falls within the range, this shows that the soil is silt(appendix 1).The specific gravity was noted and used for some calculations in other experiment e.g. hydrometer test.

4.1.4 Determination of particle size distribution

a) Results

After the test has been conducted the results were recorded, calculate the mass retained and cumulative percentage.

Table 4.3: Sieve analysis test results

Test sieve	Mass Retained (g)	Soil Retained	(%)Passing
2 mm			
1.18mm			100
600µm	8.32	14.39	85.61
425µm	6.72	11.62	73.98
300µm	4.07	7.04	66.94
212µm	3.15	5.45	61.49
150µm	2.72	4.71	56.79
63µm	6.06	10.48	46.31

b) Discussion

Wet sieving was done to avoid errors that might arise if dry sieving is done. Dry sieving depends on the compaction force applied while crushing the soil particles from the oven, therefore less compaction done ,more particles will be retained in the 2mm sieve and vise verse. Unified Classification of Soil can be classified as SM which is silty sands, sand silt mixture with more than 12% soil particles passing 425 µm and the plot is below the A-line (Day, 1999).The A-line is classified after casangrande from the liquid and plastic limit. The subdivision of this soil is Sands meaning the highest percent of soil if found on the sand region as shown in the particle size distribution chart. From the particles distribution chart figure 4.1 ,it is noted that: % Gravel=0; % Sand= 36; % Silt= 42% and clay=22%.Since gravel ≤15%, this soil is classified as Sandy elastic silt (Das, 2006). Appendix 3 shows the USDA textural classification chart that indicates that this type falls under silt loam classification. From figure 4.1 D_{10}=0.0045 mm ,D_{30}=0.04 mm, D_{60}=0.18 mm from this C_u=40 and C_c=1.975.

4.1.5 Sedimentation by hydrometer method

a) Results

The calibrations used for this experiment is provided in appendix 4 and appendix 5 for the correction factor. The table below shows the results that were computed during the experiment.

Table 4.4: Hydrometer sedimentation particle size distribution

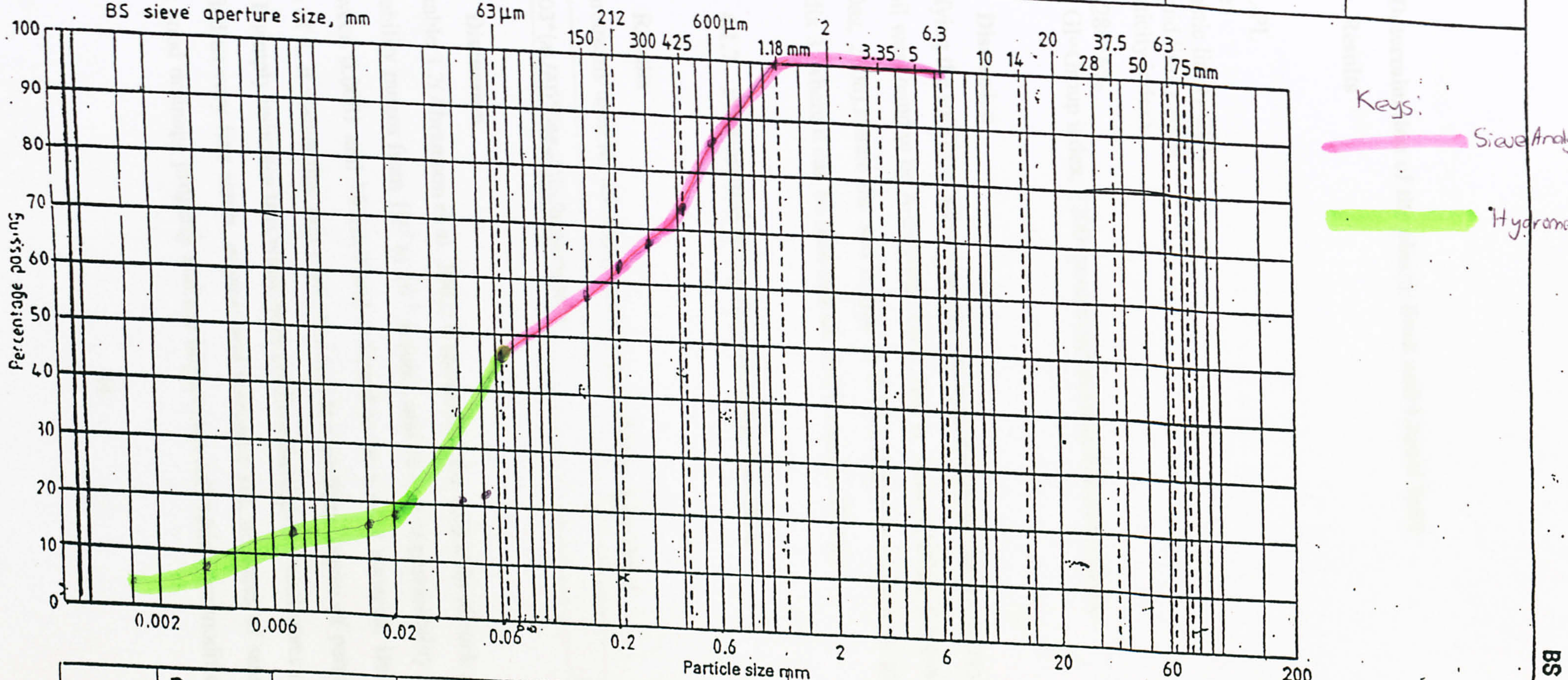
Time elapsed t (min)	$R_h'+C_m=R_h$	Effective depth Hr(mm)	Particle diameter D(mm)	$R'_h-R_o'=R_d$	Percentage finer than D ,K%	% Adjusted Finer P_A
0.5	15	199	0.0553	16.5	45.72	21.2
1	14.5	201.1	0.0393	16	44.34	20.53
2	14	203.2	0.0279	15.5	42.95	19.9
4	13.5	205.2	0.0199	15	41.57	19.2
8	12	211.4	0.0104	13.5	37.41	17.3
30	10.5	217.5	0.0075	12	33.25	15.4
120	8.5	225.7	0.0054	10	27.71	12.8
480	6	236	0.0039	7.5	20.78	9.6
1440	2.5	250.4	0.0016	4	11.08	5.1

b) Discussion

An object that is denser than a liquid will sink in that liquid and an object that is less dense than a liquid will float in it. The hydrometer will sink lower in less dense liquids than in more dense liquids. The particles in less dense solutions are not as tightly packed as the particles in more dense. More dense liquids have more particles in the same volume to help push the hydrometer up than the less dense liquids. The hydrometer will sink lower in the vegetable oil than in the water and will float higher in the salt water solution. This results were also plotted in the same logarithmic chart as the sieve analysis but indicated by different colors (figure 4.1).Hydrometer shows the settling of particles of less than 63μm.

Particle size distribution chart

Location	Soil description	Job ref.		Sample no.	
		Borehole/Pit no.		Depth	m
Test method	BS 1377 : Part 2 : 1990 : 9.2/9.3/9.4/9.6/9.7 *			Date	



CLAY	Fine SILT	Medium	Coarse	Fine SAND	Medium	Coarse	Fine GRAVEL	Medium	Coarse	COBBLES	BOULDERS

* Delete as appropriate

Operator	Checked	Approved

BS 1377 : Part 2 : 1990

Figure 4.1: Sieve analysis and Hydrometer reading

4.1.6 Determination of the plastic limit and Liquid limit

a) Results

Ip=LL-PL
Where:
PL(plastic limit)=28%
LL(liquid limit)=44%
Ip(plasticity index)

Ip=44-28=16%
Where GI=Group index, F200=percentage passing this sieve number

b) Discussion

Classifying the soil(from sieve analysis and hydrometer) by the AASHTO classification, this soil was found to be A-2-7,with GI=1.This is from equation GI=0.01(f200-15)(PI-10) (Das, 2006).Hence the soil is type A-2-7(1).This classification is also attached as appendix 3, where it can be seen that the soil is mainly silt clay.

4.1.7 Permeability using the falling head method

a) Results

Measurements used for this experiment is provided in appendix 4

$k=2.303*(aL/At)*(\log_{10}*(h_1/h_2))$(cm/sec)	0.0036

b) Discussion

From table 3.2(Abramson et al. 2002) it shows that the void ratio of such a coefficient of permeability ranges from 10^{-2} to 10^{-1} in (cm), and its soil of permeability as well since k is between 0.0001 and 10 cm/s and is found to be a sand material. Due to this range values of k, it shows that this particular soil has medium degree of permeability (Day, 1999).It simply indicates that water does not flow through the void space rapidly. This is classified as very fine sands, organic and inorganic silt, mixtures of sand silt and clay with a good drainage property and are impervious soil which are modified by effect of vegetation (Day, 1999).From appendix 9 this type of soil drains slowly and has fine sands, organic and inorganic silts.

4.1.8 XRD

X-Ray Diffraction (XRD) is a non-destructive technique that reveals detailed information about the chemical composition and crystallographic structure of natural and manufactured materials.

a) Results

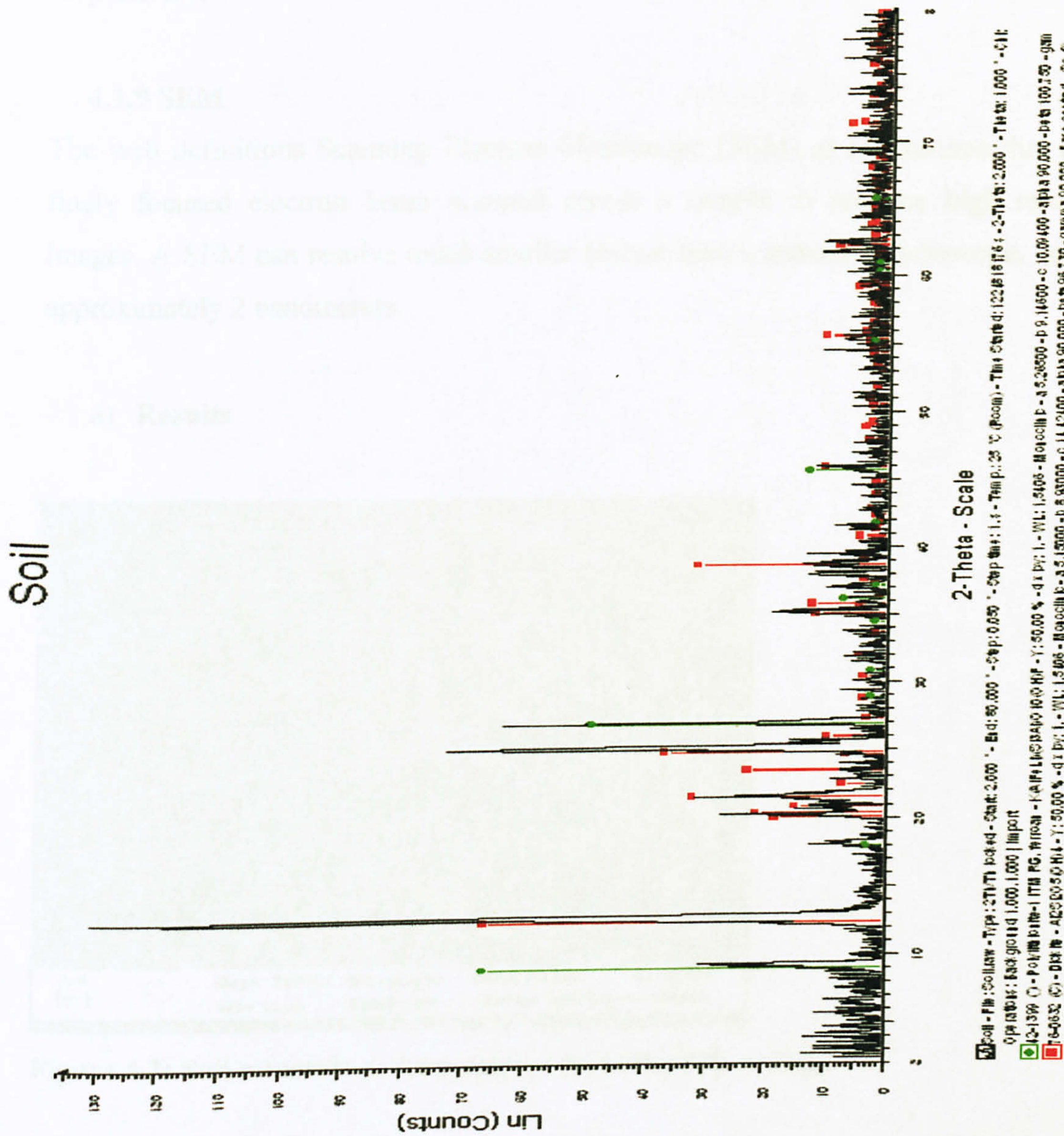


Figure 4.2: XRD

b) **Discussion**

From figure 4.2 it can be observed that there are black traces which indicates the saw soil, the green traces which are the polylithionite mineral and the red traces are that of dickite mineral. This type of soil can be described as crystalline structure of soil, (i) large traces of polylithionite ($KLi_2AlSi_4O_{10}$) this polylithionite is composed of Potassium, Lithium Aluminum, Silicate and Fluoride. This mineral falls under the Silicates class and (ii) adequate of dickite ($Al_2Si_2O_5(OH)_4$), It is kaolinate group of clay mineral (aluminum silicate hydroxide) chemically composed of aluminum, silicon, hydrogen and oxygen contributing 20.90%, 21.76%, 1.56%, and 55.78% each respectively.

4.1.9 SEM

The web definitions Scanning Electron Microscope (SEM) as microscope that uses a finely focused electron beam scanned across a sample to produce high resolution images. A SEM can resolve much smaller feature than a standard microscope, down to approximately 2 nanometers.

a) **Results**

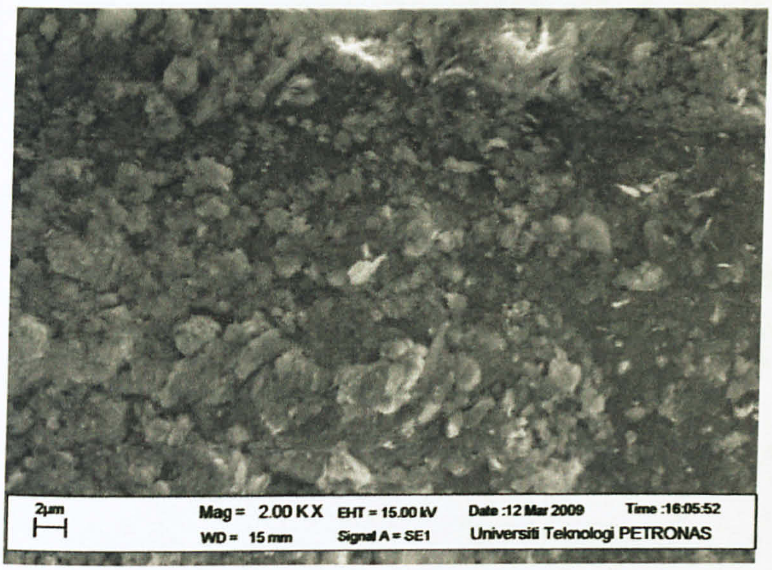


Figure 4.3: Soil minerals as they appear from the Microscope

b) Discussion

As seen from a microscope picture the particle size of the soil have diameter of 2 μm implying that it consists more of silts and a little clay , this type of soil is clay silt. This SEM simply confirms with us with the type of soil existing at the site.

4.2 Results from the previous studies by Sapari, et al. (2008)

Below is a table indicating the results that previous studies of the soil from the site in determining possible cause of the slope failure in UTP without trenching provided.

a) Results

Table 4.5: Moisture content of soil samples from different levels of the slope model without trenching provided (Nasiman el at, 2008)

Level of water (from the bottom of the tank)	Location of soil sample	Moisture content %
21cm	Layer 3(14-21 cm) Layer 2(7-14 cm) Layer 1(0-7 cm)	40.71 41.72 42.33
14cm	Layer 3(14-21 cm) Layer 2(7-14 cm) Layer 1(0-7 cm)	37.34 40.48 40.64
7 cm	Layer 3(14-21 cm) Layer 2(7-14 cm) Layer 1(0-7 cm)	13.27 21.56 35.57

b) Discussion

Soil in both Layer 1 and Layer 2 were containing moisture optimum of 40% level i.e. almost completely saturated with water. Slope failure occurred when the moisture content of the soil reached 40% Sapari, et al. (2008).

4.3 Laboratory model studies

The initial average moisture content as mentioned before was found to be 23%, then the soil was air dried for a period of two weeks to facilitate in decreasing the moisture of the soil. After two weeks the moisture content was found to be 10% before the experiments using the model was carried out and the table below shows the results that were obtained.

a) Results

Table 4.6: Moisture content of soil samples from different levels of the slope model without trenching provided and the standpipe reading

Level of water (from the bottom of the tank)	Location of soil sample	Moisture top soil %	Moisture deep soil %	Stand pipe reading (cm)
21cm	Layer 3(14-21 cm)	19	39	18
	Layer 2(7-14 cm)	21	41	11
	Layer 1(0-7 cm)	23	43	4
14 cm	Layer 3(14-21 cm)	15	40	12
	Layer 2(7-14 cm)	17	34	6
	Layer 1(0-7 cm)	19	36	3
7cm	Layer 3(14-21 cm)	12	34	5
	Layer 2(7-14 cm)	15	31	3
	Layer 1(0-7 cm)	17	29	1

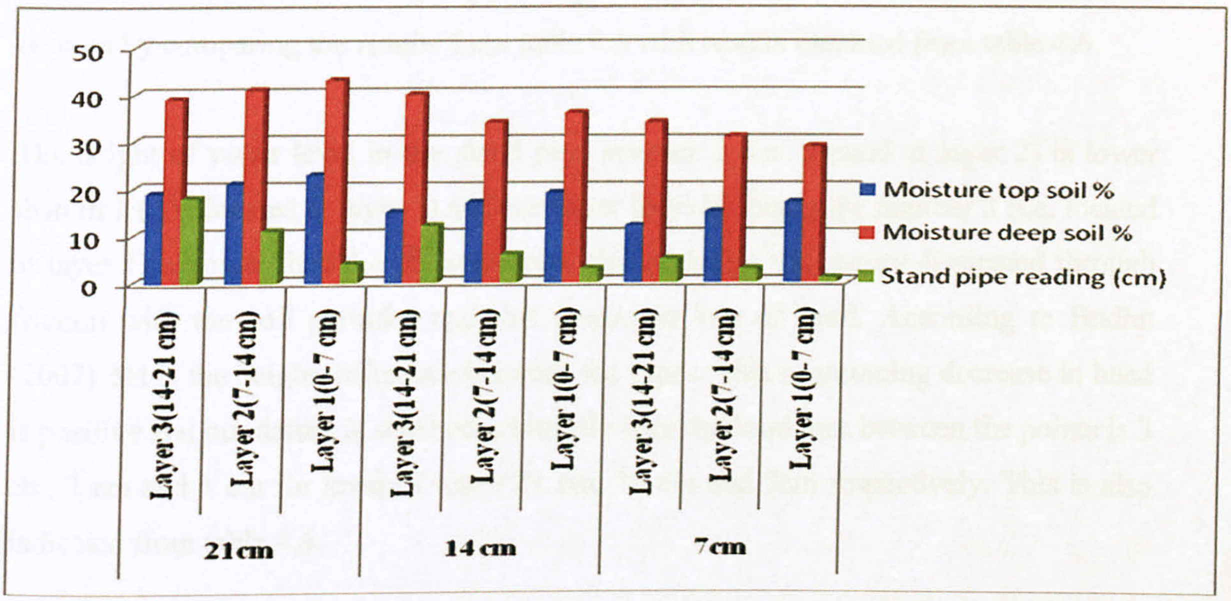
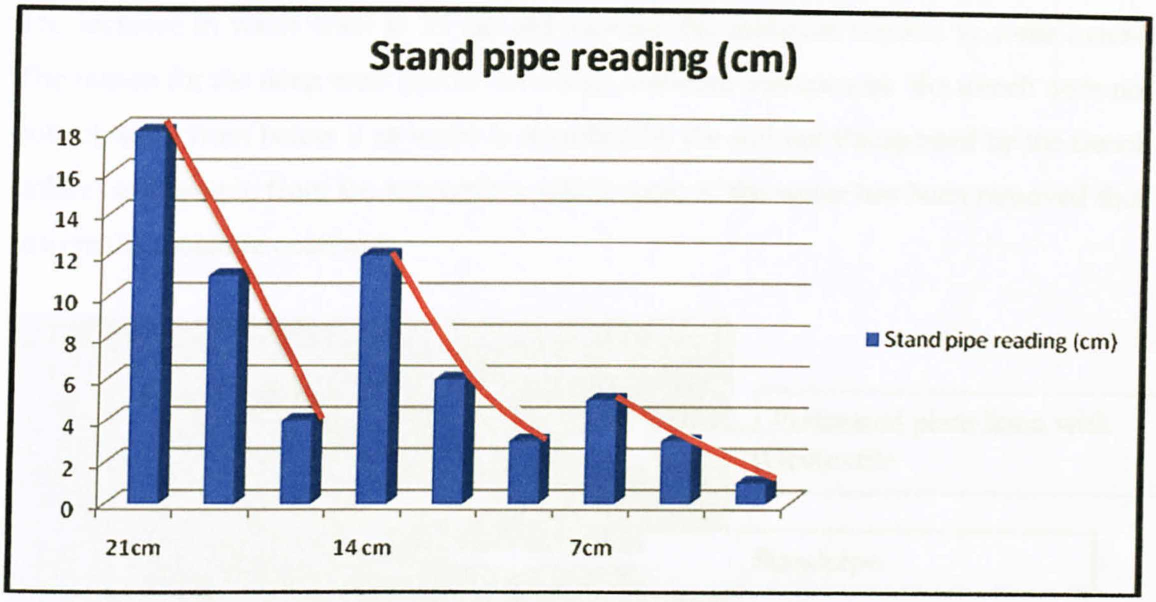


Figure 4.4: Moisture for top soil, Deep soil and the standpipes readings for different water levels of the slope model



b) Discussion

The initial air dry moisture content was 10%. Trench allows reduction of moisture content where deep most soil sample has high moisture (reaching the liquid limit)whereas the soil near the surface has moisture around 20%. Comparing this results with the results obtained from previous studies were the experiment was done without the Trench, moisture has decrease drastically. From the stand pipe reading it can be seen that the water level decreases along the slope. The difference from these two studies can be seen by comparing the results from table 4.5 with results obtained from table 4.6.

The height of water level in the stand pipe number 2 (i.e. located at layer 2) is lower than in 1 (i.e. located at layer 1),and the water level in stand pipe number 3 (i.e. located at layer 1) is lower than the 2nd stand pipe this is due to the energy dissipated through friction with the soil particles and this results in loss of head. According to Budhu (2007) ΔH is the height difference between the pipes. This is assuming decrease in head is positive and our datum is selected arbitrarily then the head loss between the points is 3 cm, 3 cm and 1 cm for level of water 21 cm, 14 cm and 7cm respectively. This is also indicated from table 4.6.

The increase in water level to 21 cm did increase the moisture content to some extent. The reason for the deep most soil to have high moisture content was the trench does not collect water from below it so water is absorbed by the soil not transported by the trench unlike soil sample from the top surface which most of the water has been removed thus lowers the moisture content

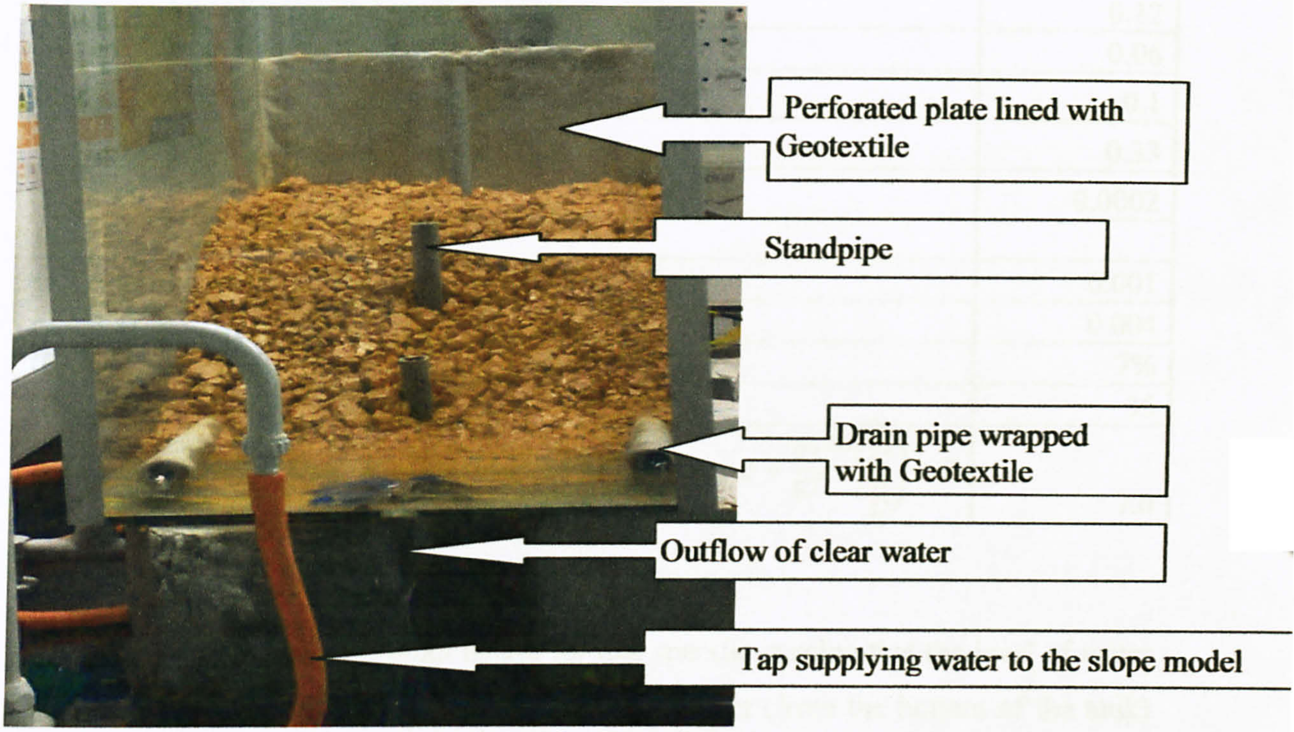


Figure 4.5: Release of clear water from the Trench

From figure 4.5, it can be seen that clear water was transported by the Trench and out of the slope model. The water was clear due to the Geotextile that is wrapped around the Trench, as said before the function of the geotextile here is to separate soil and water by allowing only water to flow through the perforated pipe. With trench provided very little amount of water seeps through the toe of the slope.

4.4 Theoretical calculation of Y_{max}

This section shows the maximum Y_{max} that water level can reach given that our slope is not sloping and using the readings while the water level of the soil is raised to 14 cm from the bottom. This Y_{max} will change depending on the value of the rate of infiltration.

a) Results

Table 4.7: Calculation of Y $_{max}$

$qz = k\Pi(h_2^2 - h_1^2) \div \ln(r_2/r_1)$	
Where:	
(K)hydraulic conductivity(cm/s)	0.004
h2=water level height-2nd stand pipe(m)	0.12
h1=water level height-1st stand pipe(m)	0.06
r2=2nd stand pipe distance from zero(m)	0.1
r1=1nd stand pipe distance from zero(m)	0.33
qz(m^3/s)	0.0002
FOR	
(q)design storm=qz*At(cm/s)	0.001
K(cm/s)	0.004
(∂)drainage slope	7%
(P)distance between pipes(cm)	35
$\text{Ymax(cm)} = \left(P\left(\frac{q}{k}\right)\left\{\left[\left(\frac{k\tan 2\partial}{q}\right) + 1 - \left[\left(\frac{k\tan\partial}{q}\right) * \left(\tan 2\partial + \frac{q}{K}\right)^{0.5}\right]\right]\right\}\right) \div 2$	7.0

b) Discussion

Y $_{max}$ is the Theoretical maximum height for this specific spacing that the level of water can rise up to. This results are for 14 cm Level of water (from the bottom of the tank) ,this is not the practical value rather the theoretical value for level(no sloping) areas. This shows that water can only go up to this level if this is exceeded slope failure can occur. Errors might have occurred while calculating the design storm. This maximum value was how ever not taken into consideration in designing the model, since we are dealing with slope model but can how ever be applicable if we dealing with level areas or for landfills.

4.5 Soil investigation Report

This part indicates results obtained from the site investigation report that was carried out by SI contractor.

4.5.1 Borehole Test

a) Results

From borehole log data, soil profiles had been prepared on determining changes in stratigraphy, and also locating bedrock using Standard Penetration Test (SPT) N value. This is for determining the soil layer so that the analysis can proceed from here.

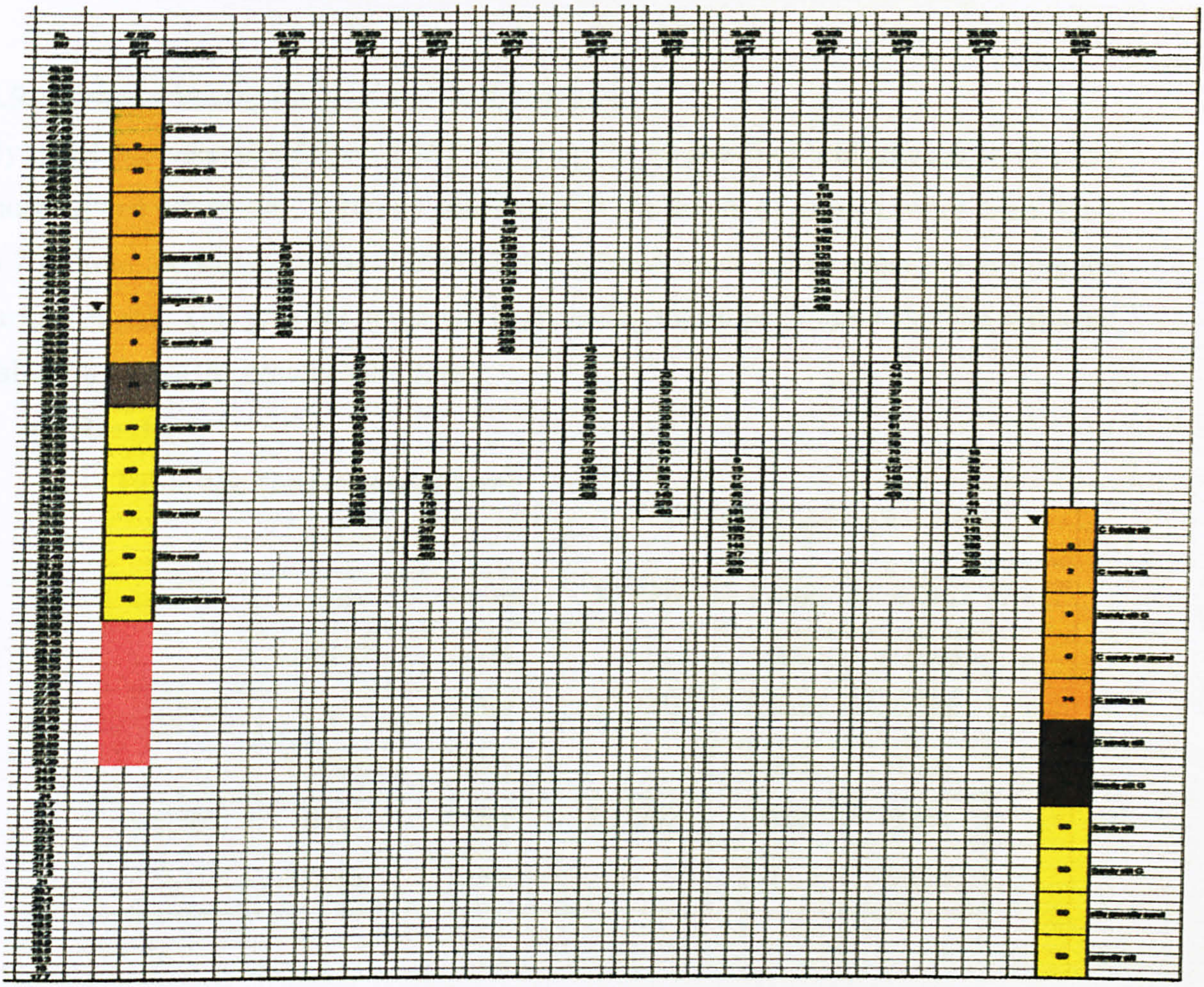


Figure 4.6: Soil profile of the slope in the study area

b) Discussion

Based on profiling data (Figure 4.6) show that the slope contains 3 layers of soil and bedrock. The soil can be identified as clay silt, clay sandy silt, and silty sand soil. They are identified based on N value of SPT:

a. Residual soil I (N=2 to 10)

b. Residual soil II (N= 10-30)

c. Residual soil III (N>30)

Looking at the soil, and comparing with "Weathering Zone Classification System Recommended by the Geotechnical Control Office (1984) for Use in Hong Kong", the soil can be classified on the zone A which is structure less sand, silt and clay.

4.5.2 Ground Water Level at the landslide area

By installing water standpipes, and monitoring the tabulated of groundwater level every morning and afternoon , it can be conclude that the height of ground water from BH-1 is 7 m from the surface while for BH-2, the ground water is just about 0.6 m from the surface of soil. This data are important in analyzing the slope failure using the software. Table 4.8 shows the groundwater level.

a) Results

Table 4.8: Groundwater level

BH-1			BH-2		
1/12/08	1.0pm	4.10m	1/12/08	1.0pm	0.7m
	5.0pm	6.2m		5.0pm	0.65m
2/12/08	8.0am	7.6m	2/12/08	8.0am	0.7m
	5.0pm	7.0m		5.0pm	0.7m
3/12/08	10am	7.0m	3/12/08	10am	0.6m
	5.0pm	7.0m		5.0pm	0.6m

b) Discussion

Based on the result, as for elevation of 15m from ground surface, the ground water level is 7 meter below the surface while for BH 2, the ground water is 0.6m below the surface. The typical groundwater level at the toe of the slope is considered to be high. The ground water level has increased and become higher because of heavy rain and due to the absence of the Trench this resulted in a big quantity of water seeping through the toe of the slope.

4.5.3 Index Properties of the soil

a) Results

Based on the laboratory test report for undisturbed and disturbed sample (Table 4.9) the index properties of the soil can be summarized as follow:

Table 4.9: Index properties of the soil

Soil Layer	Unit Weight (kN/m^3)	Drained Cohesion, C' (kN/m^2)	Drained Friction Angle, ǿ (degree)
Soil Layer I	19.8	5	32
Soil Layer II	19.8	10	35
Soil Layer III	21.66	15	37

b) Discussion

From analysis, it can be shown that the analysis for typical value of unit weight for silty soil is in the range of 18-20 kN/m^3. As the data are within the typical value of unit weight, thus it is acceptable. Silt soil has a valid cohesion number i.e. $C' \neq 0$, as shown from this results. Predominantly it consist of clay and silt soil, fine- grained particles, that sticks together whether wet or dry as cohesive soil.

4.6 Analyzing Slope stability Analysis using SLOPE/W

Analysis on slope stability is important on determine the Factor of Safety (FOS) of the critical slope failure.

4.6.1 Case 1: Failure state

a) Results

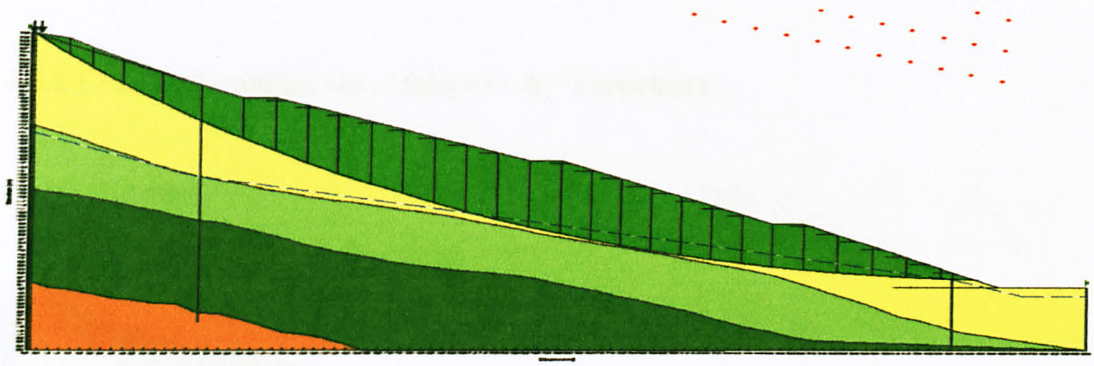


Figure 4.7: Bishop simplified factor of safety and critical slip surface for global failure

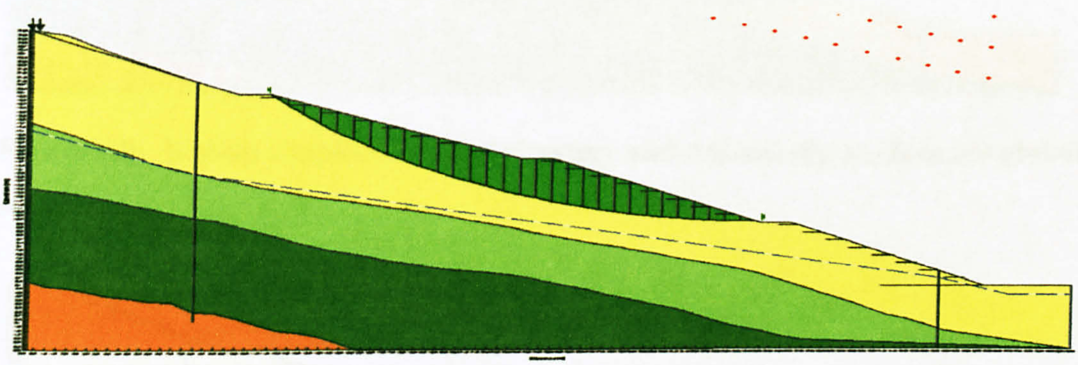


Figure 4.8: Bishop simplified factor of safety and critical slip surface for local failure

b) Discussion

This results shown in Fig 4.7 show the contour of slip surface analysis where FS = 1.092 in the state of global failure. For local failure, Fig 4.8 shows the critical slip surface of failure at FS = 1.26. Both global and local failure had been examined to analyze the possible occurrence of slip surface failure. Global failure happen when the slip surface axis is assumed to be at the end of both slope and it shows many possible global state failure while as local failure, the slip surface axis had been assumed to be on certain level of the height, as for this case, the slip surface axis had been assumed to be at the second level of the berm and the other one is at the first level of the berm. Figure 4.8 *shows the effect of groundwater level to the slope which is related to the previous* studies analysis as discussed in 4.2. Our slope failure is assumed to be that of local failure since the other part of the slope did not fail mainly slope of the 2nd tier.

4.6.2 Case 2: Remedial slope failure – by Trenching

a) Results

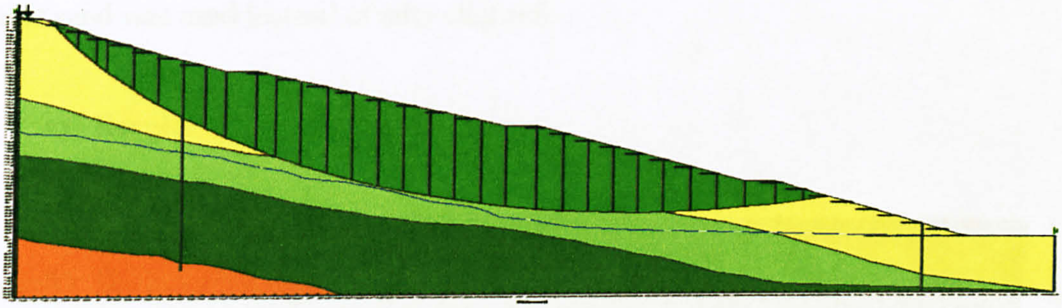


Figure 4.9: Bishop simplified factor of safety and critical slip surface for global state

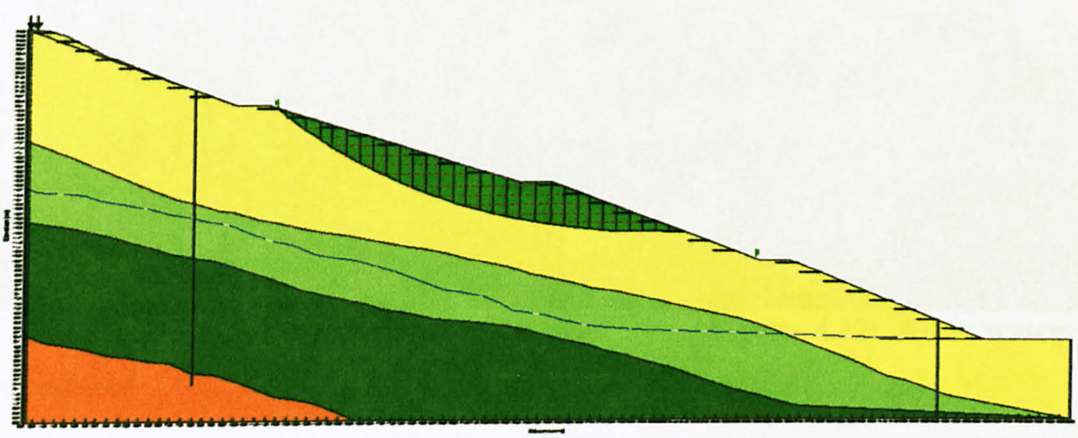


Figure 4.10: Bishop simplified factor of safety and critical slip surface for local state

b) **Discussion**

The effect of Trenching is shown in this result. Fig 4.9 shows the critical slip surface analysis after the installation of trenching which decreased the moisture content. The moisture content and groundwater level are part of the parameters needed for calculating FS. Where FS value had increased to FS = 2.187 in the state of global state. For local state, Fig 4.10 describes the critical slip surface after Trenching installation and the FS = 1.849. It shows that, when trenching had been installed at the slope for remedial action, the FS increased and it means **the slope is in the condition of safe.**

4.7 Control experiment

A control experiment was designed and operated using sand soil. All consideration made for the experiment before and procedures were the same the only difference was that sand was used instead of silty clay soil.

a) **Results**

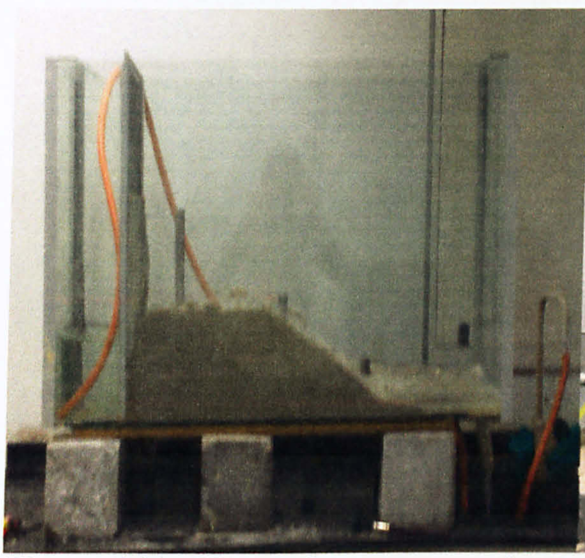


Figure 4.11: Slope after set up using sand **Figure 4.12: After water was introduced**

b) Discussion

After water was introduced into the talk the sloped failed immediately even before water had a chance to flow into the drain, it immediately over flow the surface of the slope. The reason for this failure could be that sand has too much voids and is cohesive less unlike the silty clay soil used before. The failure is shown Fig 4.12.

There are a number of methods to rectify the slope. Most of the modifications mentioned above are either not applicable or they have been done e.g. terrace and vegetation cover. However, in our point of view, it is important to implement hydraulic modification by constructing a vertical trench from the centre of concave drain at the 3rd tier downward to the first tier and finally to the roadside drain in order to prevent overflow from the third tier drain.

A downward slope trench is also proposed, used to direct surface runoff or groundwater accumulated at the centre of 3rd tier drain going downward into a roadside drain thus without eroding the soil at the slope. This will protect the slope from water erosion. Alternatively gabion bed such as those in other areas of the slope along the road. However, at the landslide area, this was not constructed.

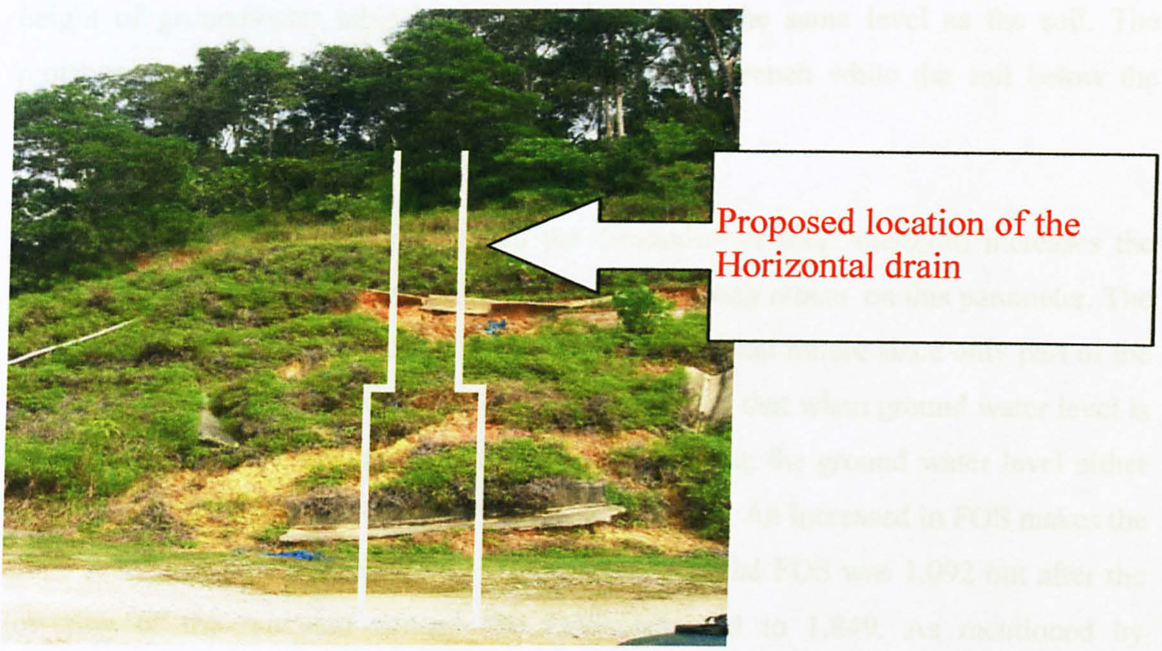


Figure 4.13: Site condition

Chapter 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The soil in UTP campus consists of mostly of sands, silt and clay. All the laboratory experiments that were conducted for the soil analysis and from the site investigation report both showed and resembled this type of soil.

Trenching lowers the moisture content of the soil therefore is potentially effective method of control of slope failure. This is supported by comparing the effect on the moisture content of soil while using and while not using a trench. The moisture content when no trench is provided ,could reach the liquid limit above 40% and while when trenching is provided the moisture content was found to be above 24%. It was found that the presence of the trench reduces the level of moisture content in the soil even when the height of groundwater table behind the slope is at the same level as the soil. The moisture content was always below 24% above the trench while the soil below the trench may reached the liquid limit of about 42%.

Lowering the moisture content lowers the Groundwater level therefore increases the Factor of Safety (FOS) since the safety dependent among others on this parameter. The type of slope failure that occurred from our site is the local failure since only part of the slope failed. Based on the analysis of the case, it shows that when ground water level is high, FOS is low and the slope is not safe. By reducing the ground water level either using horizontal drain or trenching the FOS is increased. An increased in FOS makes the slope safer thus land sliding can be prevented. The initial FOS was 1.092 but after the lowering of the moisture content the FOS increased to 1.849. As mentioned by D'Acunto. and Urciuoli (2006) that FO above 1.2 for slope can be considered as safe enough, in this case the initial FOS is not safe enough since it is < 1.2

Decreasing the amount of water in the soil decreasing the volume of voids which increases the dry density and will in turn increase soil strength.

5.2 Recommendation

The landslide occurred at Building 13 at worst can cause an accident or fatality to the road user as there are a few students/staffs that use the road or park their cars near the road. Furthermore, there is a main drain and underground utilities system under the access road that may be affected by the possible landslide and flood. The site requires attendance.

In Malaysia, occurrence of slope failure or landslide is most prominent with this type of soil. This is made worse by the fact that Malaysia has a high annual rainfall which makes movement of water on the surface and sub- surface in large quantities. Such type of areas should be carefully studied to avoid similar cases from occurring.

Laboratory equipments should be regularly maintained at good working order since it can affect experiments conducted by student. Due to this problem some of the soil characteristics could not be identified.

6. References

Abdullah C.H., Mohamad A., Yusof M.A.M., Gue S.S. & Mahmud M. (2007), "Development of Slope Management in Malaysia".

Abramson ,L.,&Lee,T.,&Sharme,S.,&Boyce,G.(1999).Slope stability and stabilization methods(2nd ed.).Jeff Faust.

Asbi Othman, Mahadzer Mahmud, Ashaari Mohammad & Mohd Jamal Sulaiman (2007),"Landmark Landslides in Malaysia".

Berilgen.MM,Computers and Geotechnics 34 (2007) 81–91.

BS 1377(British Standards Institution / 28-Feb-1990 / 38 pages ISBN: 0580176924).

Bowles J.E,1995,(5th ed.). McGraw-Hill Science.

Budhu,M.(2007). Soil mechanics and foundations(2nd ed.).John Wiley&Sons,Inc.,Hobeken.

Brunsden,D.& Prior, D.B. eds., John Wiley and Sons, New York, 1984. Slope Instability

Chigira.M ,Wangb.W, Furuyac.T , and Kamai.T . (2003). "Geological causes and geomorphological precursors of the Tsaolinglandslide triggered by the 1999 Chi-Chi earthquake, Taiwan," Engineering Geology 68 (2003) 259–273..

D'Acunto,B. and Urciuoli,G. , April 2006, Mathematical and Computer Modeling, Volume 43, Issues 7-8, Pages 754-765.

Das,B,M.(2006).Principles of Geotechnical Engineering(6th ed.). Thomson Learning College.

Day,R.M.(1999). Geotechnical and foundation Engineering, R.R Donnelley&sons company, Hightstown,new Jersey.

Fakher.A, Khodaparast M.& C.J.F.P. Jones(2006) Quarterly Journal of Engineering Geology and Hydrogeology; 2006; v. 39; issue.2; p. 189-196; DOI: 10.1144/1470-9236-05-039 .

Gue, S. S. & Tan Y. C. (2006), "Landslides: Abuses of The Prescriptive Method", Internationals Conference on slope 2006, Kuala Lumpur, 7 - 8 August 2006.

Krahn,J.(2004). Stability modeling with slope/w.An engineering methodology.

Kehew,.A,.E,(2006),Geology for engineers and Environmental science(3rd ed.). Prentice Hall, Western Michigan University.

Ludwig, D, Hilborn, R & Walters, C (1993) 'Uncertainty, resource exploitation and conservation: lessons from history.' Science, vol. 260, no. 2, p. 17.

Sapari.N, Indra S.H, Hamonangan H,Mohd A(2008)'The influence of rising groundwater on slope stability and engineering properties of soil, 2nd engineering conference on the suitable engineering infrastructure development and management, Kuching,Sarawak,Malaysia

Saxov ,S and.Nieuwenhuis, J. K eds., NATO Conference Series, Series IV: Marine Sciences, Vol. 6, Plenum Press, New York, 1982. Marine Slides and Other Mass Movements.

Turner,A,K.& Schuster,R.L., eds., Special Report 247, Transportation Research Board, National Research Council, Washington, D. C., 1996, 673 pages. Landslides Investigation and Mitigation.

Vesilind P.A&William Worrell.W,,Debra Reinhart.D,(2000),Solid waste engineering,Bill Stenquist,Pacific Grove.

Waltham.T,(2002),Foundations og Engineering Geology,(2nd ed.).Taylor& Francis Group,Abingdon.

Zaruba, Q., and V. Mencl, (1969),Landslides and Their Control, Elsevier, New York.

Appendixes

Appendix 1

SOIL TYPE	G_s
Quartz sand	2.64 - 2.66
Silt	2.67 - 2.73
Clay	2.70 - 2.90
Chalk	2.60 - 2.75
Loess	2.65 - 2.73
Peat	1.30 - 1.90

Appendix 2

Table 4.1 Classification of Highway Subgrade Materials

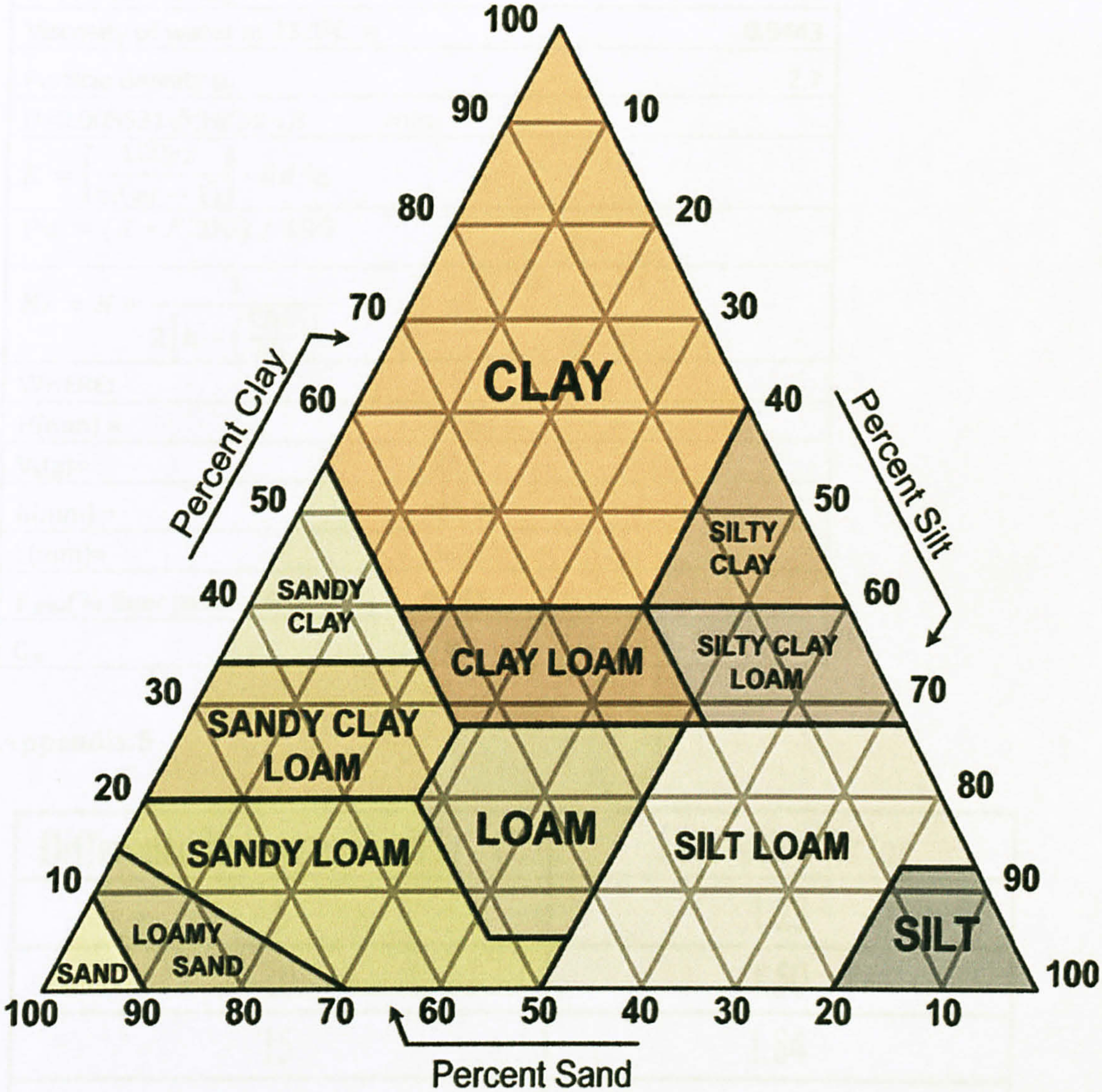
General classification	Granular materials (35% or less of total sample passing No. 200)						
	A-1			A-2			
Group classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7
Sieve analysis (percentage passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-6	A-7 A-7-5[a] A-7-6[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a]For A-7-5, $PI \leq LL - 30$

[b]For A-7-6, $PI > LL - 30$

Appendix 3


100
90
80
70
60
50
40
30
20
10
Percent Clay
Percent Silt
Percent Sand
CLAY
SILTY CLAY
SANDY CLAY
CLAY LOAM
SILTY CLAY LOAM
SANDY CLAY LOAM
LOAM
SILT LOAM
SANDY LOAM
LOAMY SAND
SAND
SILT

Appendix 4

Room temp(^{0}C)		22.6
Dry mass of soil(g)		50
Viscosity of water at 23.3^{0}C η		**0.9443**
Particle density p_s		2.7
D=0.005531$\sqrt{\eta H/(ps-1)t}$	mm	
$K = \left[\frac{100ps}{m(ps-1)}\right] * Rd\%$		
$Pa = (K * F200) / 100$		
$Hr = H + \frac{1}{2\left[h - \left(\frac{VhL}{90}\right)\right]}$		
WHERE:		
H(mm) =	71.64	
V_h(g)=	66.4	
h(mm) =	151.38	
L(mm)=	33312	
F_{200}=(% finer passing 63µm)	46.31	
C_m	**0.5**	

Appendix 5

Difference in temperature in ^{0} C.	Correction factor
5	1.23
10	1.50
15	1.84
20	2.25
25	2.76
30	3.35
35	4.10
40	5.00

Appendix 6


© 2009 Google
Image © 2009 DigitalGlobe
© 2009 AND
© 2009 Tele Atlas
elev 111 ft
Eye alt 2525 ft
Google
UTP building 13
Slope failure.

Appendix 7

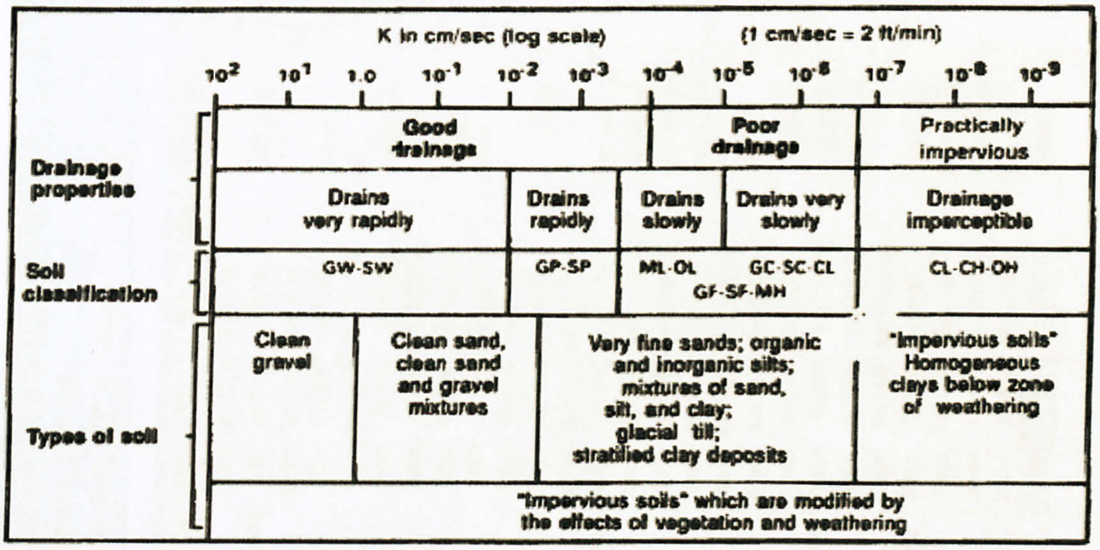
Appendix 8

Table 2. Classification of clay

Undrained shear strength, kPa	Designation
<12.5	Very soft
12.5 - 25	Soft
25 - 50	Medium stiff
50 - 100	Stiff
> 100	Very stiff

Appendix 9

Table 6-22. Drainage characteristics of soil



Appendix 10

Microsoft Project - lowani

Fri 5/29/09

	Task Name	Duration	Start	Finish	Predecessors	Resource Names
1	**Selection of Project Topic**	**1 day**	**Mon 7/21/08**	**Mon 7/21/08**		
2	Control of Ground water movement for slope stability by Trenching	1 day	Mon 7/21/08	Mon 7/21/08		FYP Supervisor
3	**Preliminary Research Work**	**35 days**	**Tue 7/22/08**	**Mon 9/8/08**	2	
4	Literature Review	6 days	Tue 7/22/08	Tue 7/29/08		Journals
5	Theory on Landslide	6 days	Wed 7/30/08	Wed 8/6/08	4	IRC
6	Historical cases	6 days	Thu 8/7/08	Thu 8/14/08	5	Web Structures
7	Cases of Landslide	6 days	Fri 8/15/08	Fri 8/22/08	6	IRC
8	Groundwater	6 days	Mon 8/25/08	Mon 9/1/08	7	Journals
9	Factor of Safety	5 days	Tue 9/2/08	Mon 9/8/08	8	Journals
10	**Submission of Preliminary Report**	**7 days**	**Tue 9/9/08**	**Wed 9/17/08**	9	
11	Joint Submission of Proposal and Progress Report	7 days	Tue 9/9/08	Wed 9/17/08		FYP Coordinator
12	**Project Work FYP1**	**48 days**	**Thu 9/18/08**	**Mon 11/24/08**	11	
13	Collection of soil sample from site and determine soil properties anc	15 days	Thu 9/18/08	Wed 10/8/08		Site Visit and Labc
14	Analyzing slope stability using SlpeW	33 days	Thu 10/9/08	Mon 11/24/08	13	MATHEMATICAL
15	Submission of FYP1 Interim	7 days	Tue 11/25/08	Wed 12/3/08	14	FYP Coordinator
16	FYP1 Oral Presentation	5 days	Mon 11/3/08	Fri 11/7/08		FYP Coordinator
17	Break	60 days	Mon 11/10/08	Fri 1/30/09	16	
18	**Project Work FYP2**	**59 days**	**Mon 2/2/09**	**Thu 4/23/09**	17	
19	Development of a Trench in the Laboratory	10 days	Mon 2/2/09	Fri 2/13/09		FYP Supervisor
20	Determine the effect of the Trench(drain),on the moisure content o	15 days	Fri 4/3/09	Thu 4/23/09	19	CALCULATION
21	Submission of Progress Report	7 days	Fri 4/24/09	Mon 5/4/09	20	FYP Coordinator
22	**Continuation of Project Work FYP2**	**37 days**	**Tue 5/5/09**	**Wed 6/24/09**	21	
23	Re do experiment,Compliation and Correlation of Result Analysis	37 days	Tue 5/5/09	Wed 6/24/09		FYP Supervisor
24	Poster Exhibition	7 days	Wed 4/8/09	Thu 4/16/09		FYP Coordinator
25	Submission of Project Dissertation (Soft Bound)	5 days	Fri 4/17/09	Thu 4/23/09	24	FYP Coordinator
26	Oral Presentation	25 days	Fri 4/24/09	Thu 5/28/09	25	Examiners
27	Submission of Project Dissertation (Hard Bound)	1 day	Fri 5/29/09	Fri 5/29/09	26	Examiners